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Experimental study of water hammer in buried PVC and Permastran pipes

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EXPERIMENTAL STUDY OF WATER HAMMER
IN BURIED PVC AND PERMASTRAN[®] PIPES

by

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Supported by

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INTRODUCTION

The magnitude of pressure waves and their velocity, generated by rapid changes in the flow conditions at a point such as the closure of a valve, are well understood, and the theory to predict both the velocity and the pressure of single waves is well developed (see Streeter and Wylie, 1967). Such hydraulic transients are commonly referred to as water hammer, since water is the fluid involved. Most of the earlier tests of water hammer pressures and velocity have been conducted in pipes constructed of metal or concrete, the common material used for construction of pipe. These materials are rigid compared to some of the materials used in pipe construction today such as polyvinylchloride (PVC).

Tests conducted late in 1969 by the Research and Engineering Center of Johns-Manville Corporation cast doubt on the validity of applying the classical water hammer equations to transient phenomena in more flexible pipe such as PVC pipe. As a consequence of these tests, Johns-Manville Corporation funded a laboratory study by the Utah Water Research Laboratory to measure the magnitudes of water hammer pressure waves, their velocities, as well as the expansion of the pipe walls under these transient pressures (see Watters, 1971). These tests were conducted on both 4 inch and 6 inch PVC pipe with initial velocities in the pipes ranging from 2 to 10 feet per second and the results indicated agreement between actual pressure waves and those predicted by the classical equations. The small discrepancy between the two could easily be attributed to air entrainment or other effects not accounted for by the theory. All of these tests were conducted with the pipe laying on the surface with only point supports to prevent lateral buckling.

Because PVC pipe is flexible and because there is a relationship between the pressure wave velocity and the flexibility of the pipe walls,

providing the pipeline has expansion joints throughout its length. If the pipeline is anchored to prevent any axial movement, the equation which gives c_1 is,

$$c_1 = \frac{2e}{D} (1+\mu) + \frac{D(1-\mu^2)}{D+e} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

After knowing the pressure wave velocity, or computing it from Eq. 1, the magnitude of the pressure increase of this wave for instantaneous valve closure can be predicted from the equation,

$$\Delta P = \rho a V_o \left(1 + \frac{V_o}{a} \right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (3a)$$

in which ΔP is the pressure increment in psf, V_o is the initial velocity in the pipe prior to valve closure, and the other terms are as previously defined. Since V_o/a is small in comparison to 1, Eq. 3a is generally used as,

$$\Delta P = \rho a V_o \quad . \quad . \quad . \quad . \quad . \quad . \quad (3b)$$

OBJECTIVES

The water hammer testing program included series of tests on 6-inch Johns-Manville Class 160 R-T PVC pipe (rated at 150 psi). Also similar series of tests were completed on Johns-Manville PERMASTRAN[®] pipe (also rated at 150 psi), which is an epoxy-fiber glass overwrapped plastic pipe. These tests for both types of pipes were for buried conditions in which soil is well-compacted (approximately 90 percent of modified proctor density). Since the previous tests (see Watters, 1971) for unburied conditions used only PVC pipe, water hammer waves were also measured in unburied PERMASTRAN[®] pipe. The objectives of the study are:

1. To measure the pressure wave velocity in the buried pipes.

2. To measure the increases in pressure in the pipes due to the pressure wave.
3. To compare the measured quantities from (1) and (2) with that predicted from theory.
4. To evaluate the effects of the surrounding soil by comparing the test results for buried conditions with those for unburied conditions, and to determine whether these effects change with the passage of repeated pressure waves.

EXPERIMENTAL APPARATUS

The pipe was buried in a fine sandy silt material (referred to as blow sand) in a 3-ft x 3-ft channel in the hydraulic laboratory portion of the Utah Water Research Laboratory. The components of the test facility which were used for the tests and measurements were essentially the same as those used in the earlier tests (Watters, 1971), but are described below for completeness.

Basic Flow System

The flow system used to perform the tests is a closed-loop system, depicted by the schematic diagram in Fig. 1. The pump which supplies the water is a variable-speed, ten-stage vertical turbine pump capable of heads up to 600 feet and flow rates up to 1000 gpm. The speed control of the pump was adjusted so that the pressure in the pipe at the pump, prior to the valve closure, was in the range of 25 to 50 psi. The higher pressures were used for the larger velocities.

From the pump the water supply was directed to the pipe being tested through a larger line which reduced to an 8-inch pipe line several feet in front of the test pipe. Several elbows and valves exist between the pump and test pipe. The pipe being tested consisted of a 120 foot section with two pressure transducers approximately 100 feet apart inserted through the pipe walls. Immediately downstream from the

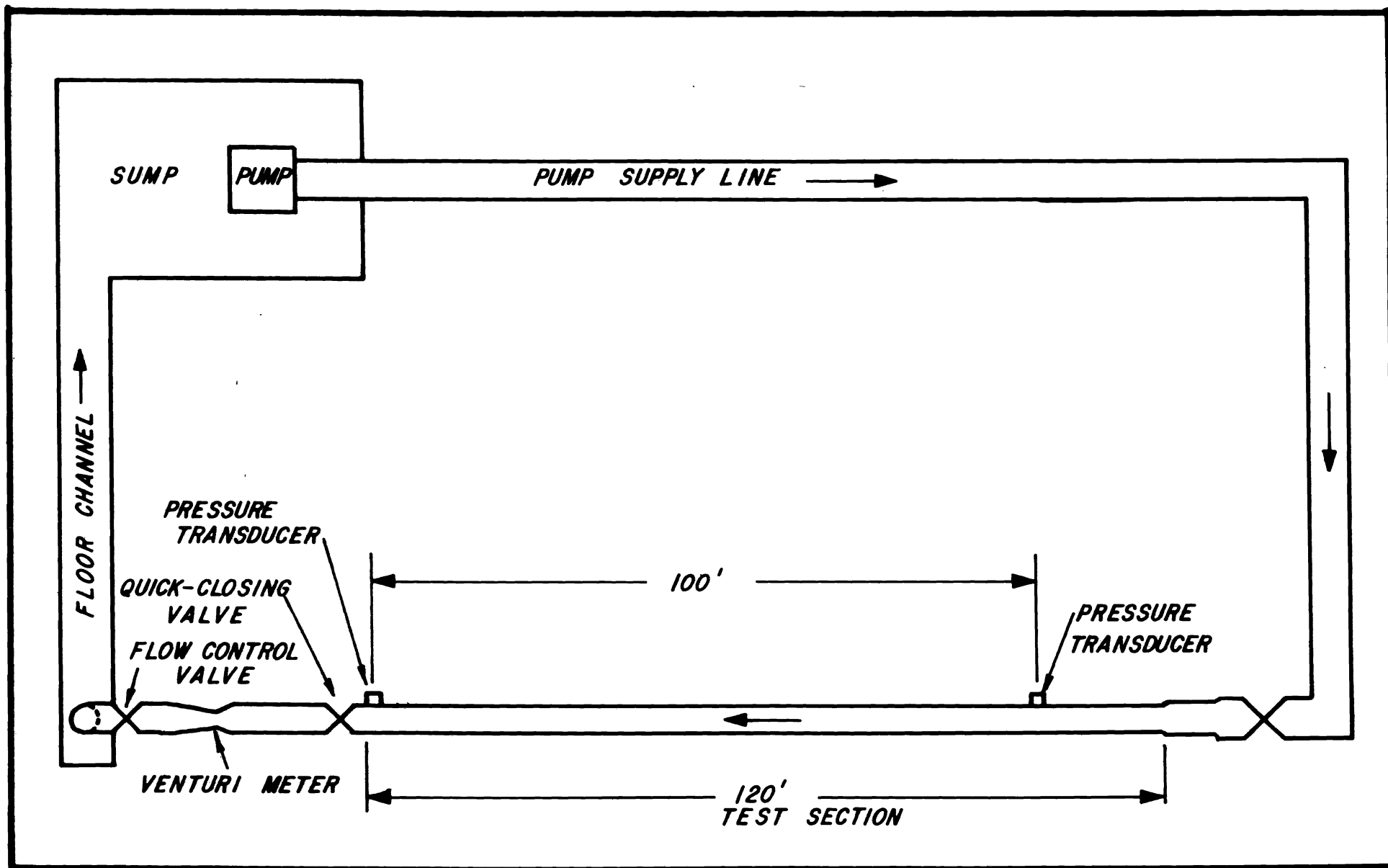


FIG. 1. SCHEMATIC DIAGRAM OF THE BASIC FLOW SYSTEM.

second transducer the flow system contained a quick-closing 6-inch gate valve. Downstream from this valve was a venturi meter connected to two manometers, one containing a blue fluid with a specific gravity of 1.65 and the other containing mercury. Below the venturi meter the line contained the valve which was used in conjunction with the pump pressure to control the flow rate and velocity in the test pipe. The venturi meter was calibrated in place during the tests described in the last report, and since the piping immediately adjacent to the venturi meter was the same in these tests as for the earlier tests, the earlier calibration curves were used.

The first quick-closing valve used was a 6-inch gate valve, consisting of two gates separated by hinges at the top and wedges at the bottom. Upon complete closure, the wedges hit the bottom and were forced upward, thus spreading the two gates each against their seats. This valve was powered by a 4-inch diameter air cylinder with a 4-inch stroke. The air supply for the air cylinder came through a solenoid air valve which was electronically controlled in coordination with the recorder and other test equipment. During the first preliminary tests this valve either failed to close completely or its closure time was too long. Subsequently, an auxiliary air compressor was used to provide air at 160 psi instead of the laboratory air supply which is approximately 80 psi. In addition, an air supply tank was added just in front of the solenoid air valve to insure that an adequate volume of air was available. The closure time was still too long, so the valve was dismantled and inspected. It appeared that the inertia of the wedges due to the rapid closure might have prematurely spread the two gates against their seats sufficiently tight to delay complete closure. Stays were then inserted in the gates to prevent the wedges from spreading the gates further apart than required for a snug fit between the seats. At the same time, two small switches were placed on the housing of the valve and an arm was attached to the stem of the valve. A voltage was placed across the

switches and the signal taken to a channel of the Visicorder. The closing of the top switch indicated the beginning of valve closure, and after the valve stem had moved 1/2 inch down the switch was again opened. When the valve was 1/2 inch from being completely closed the lower switch closed, and it opened again when the valve was completely closed. Data from these switches made it possible to determine the time required by the valve, respectively, to close the first 1/2 inch, the next 3 inches and the final 1/2 inch. These data are given with the test results on the PVC pipe. Before testing the PERMASTRAN[®] pipe, the switches were replaced by a capacitance meter which provided a continuous record of the valve closure vs. time.

The data obtained from the switches indicated that the valve closure times were too long for the last 1/2 inch of movement, particularly at the higher initial flow velocities. Therefore, the double gated valve was replaced with a single gate valve. The new valve also tended to hesitate for 0.1 of a second or more during the last 1/2 inch of closure, even at lower velocities. To speed up the closure time another solenoid air valve, with large air passageways was inserted in the air line, but with very little increase of closure speeds. Finally, the second valve was again dismantled, grease fittings inserted through both sides of the valve and a small groove machined in the seats of the valve as a passage to carry the grease. When the valve was kept well lubricated its closure times were acceptable as can be noted from the tabular test data, given later in this report.

Buried Pipe Conditions

Before installing the test pipe in the channel, approximately 6 inches of soil was placed in the bottom of the channel and compacted to approximately 90 percent modified Proctor density. The pipe was then placed on this fill. Additional soil was placed on top of the pipe and compacted by a hand operated compacter in layers of approximately 3 inches, until the total depth of compacted soil above the pipe was

1 1/2 feet. Hand tamping of the soil immediately around the sides of the pipe was used to help insure good compaction all around the pipe.

The depth of 1 1/2 feet of compacted soil above the pipe was decided upon in consultation with Dr. Reynold Watkins, a soil mechanics specialist at Utah State University. Dr. Watkins indicated that he did not believe increasing the depth beyond this amount would increase the support that the expanding pipe would receive from the fill material. Furthermore, he indicated that should this depth provide a significantly smaller amount of support than a deeper fill would, then this could be determined upon completing some water hammer tests, and observing whether any cracks occurred in the soil above the pipe. No such cracks were observed, except when a section of pipe split open during a test as described later. Therefore, it is believed that this placing of the material adequately represents installations in which the pipe is buried in well compacted fills.

Samples of the fill material were taken to the laboratory where modified Proctor compaction tests were performed at different moisture contents. The results from these tests are plotted on Fig. 2.

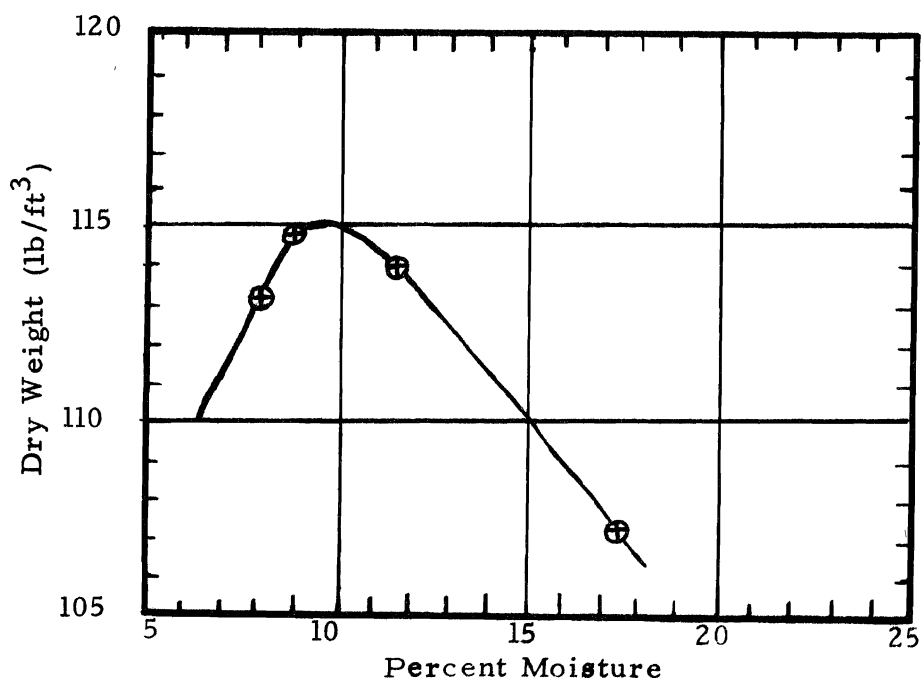


Fig. 2. Dry density-moisture content curve for blow sand used as fill around the test pipe.

Sand cone tests were performed to determine the density being achieved in compacting the fill around and over the test pipe for the first and second series of tests on the PVC pipe. The results from these tests are contained in Table 1. The first compaction achieved densities about 90 percent of Proctor, and for the second compaction around 95 percent of Proctor was achieved. The majority of the first compaction was done by a vibratory-plate, power driven, hand-operated compacter. The fill was compacted for the second series of tests primarily with a vibra-hammer type, power driven, hand-operated compacter. Since the vibra-hammer type compacter achieved greater densities, it was used in compacting the fill for the two series of tests on the buried PERMASTRAN[®] pipe. The percent compaction being achieved, however, was not determined for the two compactations burying the PERMASTRAN[®] pipe. Since the moisture conditions of the soil were maintained near optimum, it is believed that these compactations were above 90 percent and probably greater than the second time the soil was compacted around the PVC pipe.

Pressure and Wave Velocity Measurements

The pressure at the two points approximately 100 feet apart were measured by two TYCO AB-200 strain-gage pressure transducers. The gages were rated at 0-200 psi, but capable of pressures of 400 psi without damage. The transducers were excited by the same specially designed and built amplifiers that were used in the previous tests, except that a different power supply was used which had 12 1/2 volts instead of 10, and additional resistors were placed in the circuits to provide for better balancing of the bridges. Therefore, the transducers had to be calibrated again. Also, as described later, one of the transducers failed and a new one was acquired to replace it requiring still a different calibration curve.

The signals from the transducers were transmitted to the recorder and the record trace, in combination with the calibration curves (which

Table 1. Results from density testing of compacted fill

COMPACTION TESTS

First compaction for tests on PVC pipe Second compaction for tests on PVC pipe

Soil Sample:	Silty Sand	Silty Sand	Silty Sand		
Test No. :	I-A	I-B	I-C	II-A	II-B
Location:	20' downstream of upperstream transducer	60' beyond upper transducer	10' beyond upper transducer	30' beyond Channel I	70' below Channel I
Date	12-23-71	12-23-71	12-24-71	1-21-72	1-21-72
Tested By:	BSB & GRL	BSB & GRL	BSB & GRL	BSB	BSB
1. Wt Cont & Soil in lbs.	6.820	9.050	7.980	9.085	8.375
2. Wt Cont in lbs.	.726	.690	.692	.737	.682
3. Wt of Wet Soil #1-#2 = in lbs.	6.094	8.360	7.288	8.348	7.693
4. Vol of Test Hole #8/Density = Ft ³	.0543	.0715	.0628	.0692	.0624
5. Wet Density #3/#4 = lbs/ft ³	112.1	117.0	116.0	120.5	123.2
6. Wt Cont & Sand in lbs. BEFORE	14.54	13.83	14.02	13.10	12.46
7. Wt Cont & Sand in lbs. AFTER	9.70	7.63	8.51	4.18	4.09
8. Net Wt of Sand Used (6-7)-(St of sand in cone) in lbs.	4.84 .54 4.31	6.20 .53 5.67	5.51 .53 4.98	8.92 3.31 5.61	8.37 3.31 5.06
9. Wt Cont & Wet Soil in grams	134.10	160.96	161.86	190.64	216.67
10. Wt Cont & Dry Soil in grams	123.50	146.67	149.13	176.30	199.75
11. Wt of Water 9-10 = wt in grams	10.60	14.29	12.73	14.34	16.92
12. Wt of Cont in grams	36.38	35.90	37.04	36.00	36.51
13. Wt of Dry Soil 10-12 = wt in grams	87.12	110.77	112.09	140.30	163.24
14. % of Moisture - W 11/13 = in%	12.2%	12.9%	11.4%	10.2%	10.4%
15. Dry Density 5/(1+11)=lbs/ft ³ 100	99.9	103.7	104.1	109.2	11.5
16. % of Compaction #15/Proctor = %	86%	90.1%	90.6%	95%	96.9%

Comments: Sand density = 79.4 lbs/ft³
Volume of Plate = 0.0067 ft³

Sand density = 81.1 pcf

were all linear at least over major ranges of pressure), were used to obtain the pressure. Measurement, from the recorder output, of the time required for the pressure pulse to travel from the downstream transducer to the upstream transducer, provided a value of the wave velocity.

Recording System

The signals from the pressure transducers were recorded on a Honeywell 1108 Visicorder. The Visicorder is a 24 channel light-beam oscillograph, of which only four channels were used. A schematic diagram of the circuitry used for the tests is shown in Fig. 3. The galvanometers in the Visicorder which deflect the recording light beams respond to variations of current, whereas the signals from the pressure transducers through the specially designed bridge amplifier are voltage signals. These voltage signals were directed through a galvanometer amplifier, which converted the voltage signals to current signals, and this output was put directly into the Visicorder.

Both the Visicorder and the solenoid air valve used to operate the quick closing pipe valve were operated by the same manual switch. When this switch was first closed, the Visicorder started and, an instant later after the recorder was up to full operating speed, current was directed to the solenoid of the air valve to close the water valve downstream from the test pipe. Two separate manually-operated switches were used to turn off the Visicorder and open the valve.

As described later, it became desirable to measure the rate of valve closure. This measurement was achieved by mounting two switches on the stationary support of the valve and attaching a trip arm to the movable stem of the valve's gate. A voltage was placed across the switches and connected to separate bridge amplifier channels. As the valve closed, the trip arm mechanically activated the switches. Since the trip arm on the stem was 1/2 inch wide, this arrangement permitted

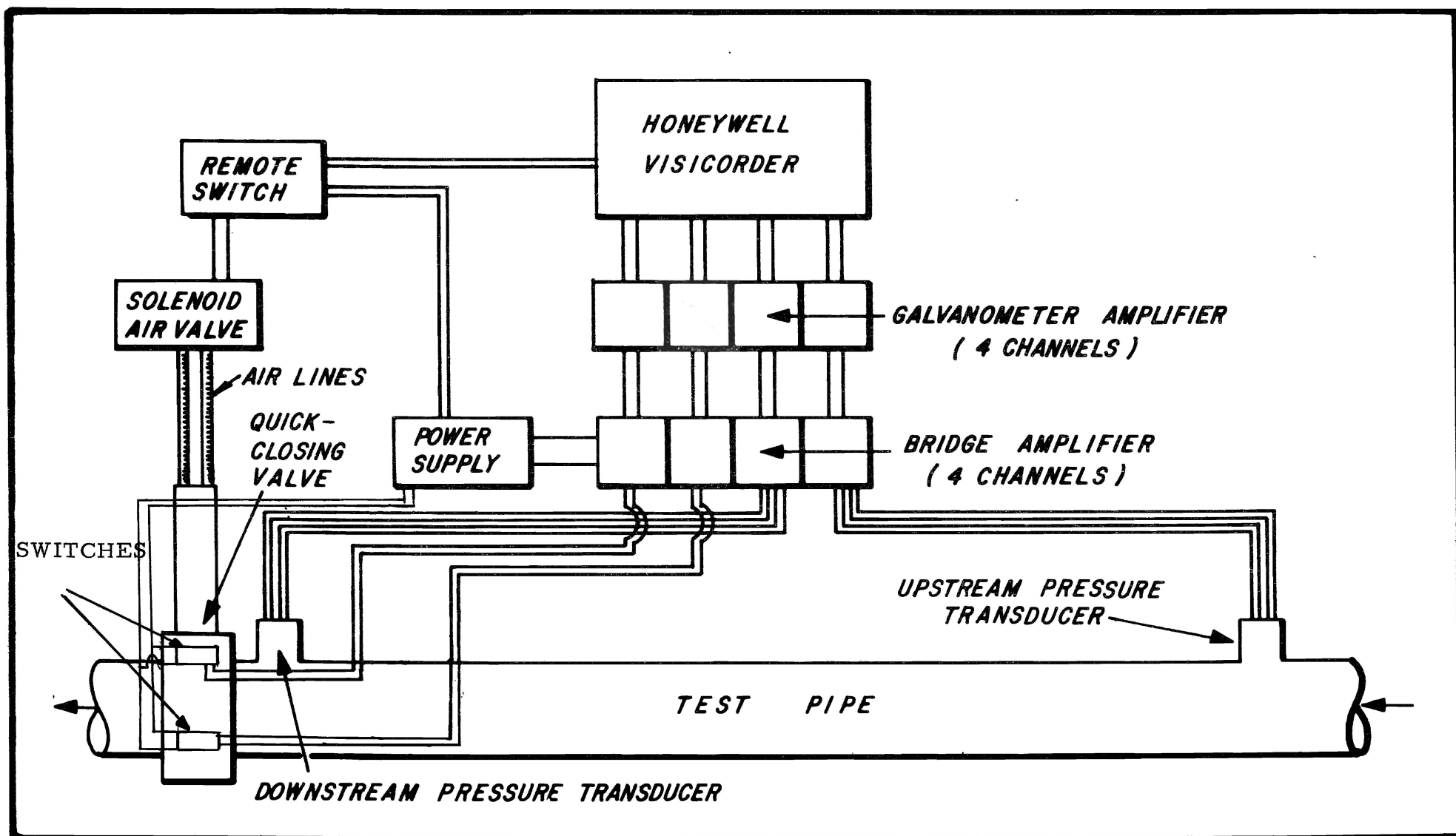


FIG. 3. SCHEMATIC DIAGRAM OF THE RECORDING SYSTEM.

timing how long it took the valve to travel the first 1/2 inch, the next 3 inches and the final 1/2 inch. This switch arrangement is shown on Fig. 3.

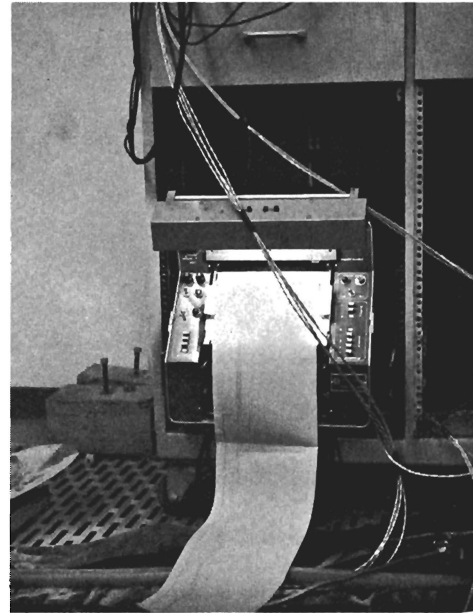
Capacitance Meter for Measuring Valve Closure

After the tests had been completed on the PVC pipe, it was decided that a more complete record of the valve's closure times would be desirable. It was subsequently found out to be unnecessary, because shorter valve closure times actually occurred for these later tests. The test results on the PERMASTRAN[®] pipe, therefore, contain a more complete record of valve closure characteristics. This record was obtained by use of a "Decker Delta Unit Model 902-1," which is an instrument designed to convert minute changes of capacitance into large analogous output voltages. The changes in capacitance as the valve moved were created by mounting two small plates of steel, each 2 inches wide by 3 inches long and separated by about 1/2 inch on the stationary support of the valve. Another steel plate 2 1/2 inches wide by 3 1/2 inches long was attached to the movable stem of the valve. The movable plate had a clearance of approximately 3/16 inch from the stationary plates. This plate arrangement constituted a variable capacitance under valve movement. The resulting output voltage from the Decker Delta unit was supplied to one of the channels of the bridge amplifier. The device was calibrated by placing the valve in a number of positions and reading the voltage output on the meter on the unit. A continuous and complete record of valve closure during a water hammer test was thereby accomplished.

The photograph in Fig. 4a shows the valve assembly with a side view of the plates which constitute the variable capacitance device. Also in Fig. 4 photographs are given of the Visicorder (4-b), and the test lay-out (4-c) and (4-d). Fig. 4-c shows the pipe after it has been buried. This photograph, which was taken facing in the downstream direction, also



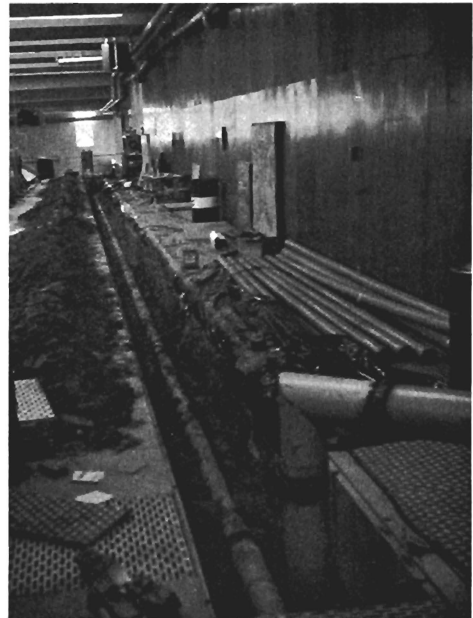
(a) Valve assembly



(b) Visicorder used to obtain data charts



(c) Channel containing buried pipe



(d) Test pipe with fill excavated to pipe level

Fig. 4. Photographs showing water hammer test facilities.

shows the vibra-hammer compacter. Fig. 4-d shows the pipe after the fill above it had been excavated.

EXPERIMENTAL PROCEDURE

For each test, the proper velocity was established in the test pipe by opening or closing the flow-control valve downstream of the venturi meter until the differential manometer showed the correct reading for that given velocity. As the test velocity increased, it was necessary to increase the speed of the variable speed pump so that the pressure was maintained in the range of 25 to 50 psi. (The higher pressure was used at higher velocities to prevent extensive column separation from occurring.) Next the manual switch was pressed which turned on the Visicorder and an instant later closed the quick closing valve. The recorder was stopped after approximately 1 to 2 feet of record was acquired, and then the valve was opened by pressing the appropriate switches. From the Visicorder record from each test, the following data were determined: (1) the time required for the pressure wave to travel between the two pressure transducers, (2) the incremental pressure in the pipe at the position of the downstream pressure transducer, (3) the incremental pressure in the pipe at the position of the upstream transducer, and (4) data giving the rate of valve closure.

Since the experimental equipment set-up was essentially the same as for the earlier tests reported by Watters (1971), it was decided that the tests on the PVC pipe would be performed, and then the calibration of the pressure transducers verified before proceeding with the tests on the PERMASTRAN® pipe. The last test completed on the PVC pipe with an initial velocity of 10 fps caused the last section of pipe and one transducer to fail. Consequently, the calibration of the downstream transducer could not be verified. An examination of the failure indicated that the pipe had split in a nearly axial direction over its full length, in a

position of approximately 15° from the bottom. In this position it was most difficult to compact the fill and, consequently, the density was probably less than that achieved at other locations around the pipe. The expansion of the broken pipe caused the fill to have two parallel cracks which were visible at the surface for the full length of the broken pipe section.

It is not possible to determine the magnitude of pressure which caused this rupture. Evidence indicates that the rupture occurred sometime after the second pressure wave arrived at the downstream transducer and after the Visicorder record had stopped. The second positive pressure wave went slightly off the Visicorder record; and, if the pressure transducer reading is still linear in this range, the pressure was approximately 260 psi (the pressure increment equals 210 psi). Prior to the valve closure the pressure at the pump, as indicated by its gage was 50 psi, but after the valve was closed the pressure at the pump increased to 255 psi as observed by an individual standing by the pump. If the speed of the pump did not change with the increased pressure, the pump characteristics would indicate if the velocity in the pipe changed from 10 fps to 0 fps the pressure would increase from 50 psi to approximately 200 psi. It is likely, however, that a couple of reflected transient pressure waves combined to cause an instantaneous pressure much higher than this. A further indication of this occurrence is that the downstream pressure transducer, which was inserted in the pipe that broke, also failed.

To determine the nature of the failure, the transducer was sent to the TYCO Company for analysis. Since the transducer was inundated under wet soil for several days due to the pipe failure, it was not known whether excessive pressures or moisture caused the failure. Suspicion of the latter was reinforced when the signal output ceased completely upon oven drying the transducer for 48 hours. However, the TYCO report indicated excess pressures caused the failure. In part, their

report reads: "We find that both semiconductor gages are broken; also the element is bent. This indicates a severe overload." The specifications of the transducer which failed stated that it is rated for a 0-200 psi range and that no damage will result up to 400 psi. The specifications also indicate that the transducer can withstand 1000 psi without bursting. Consequently, the instantaneous local pressure very likely was well in excess of 400 psi.

If as suggested above, the failure was caused by the superposition of two or more positive pressure waves, then conditions of this test were precisely those needed for this to occur. During the other tests, this combination did not occur, at least not until after viscous action had substantially reduced the peak pressure increments. Many sources of reflections for pressure waves exist in the system in addition to the valve at the downstream end. At the upstream end the test pipe is preceded by an 8-inch diameter pipe which causes partial reflection. Several 90° elbows exist between this enlargement and the pump, which would reflect the pressure wave partially. Finally, the pump-sump arrangement would cause a reflection. Since these several reflected waves could travel with somewhat different velocities due to the local condition at the time, it is not difficult to visualize how two or more could easily combine instantaneously at a point.

WATER HAMMER TESTS

Six series of water hammer tests were completed (see Fig. 5), plus a number of preliminary tests necessary for the adjustments to achieve more satisfactory valve closure times. Three of these were for buried 6-inch PVC pipe, two for buried 6-inch PERMASTRAN® pipe, and one for unburied 6-inch PERMASTRAN® pipe. Between the first two series of tests on the PVC pipe, and the first two series of tests on the buried PERMASTRAN® pipe, the fill was excavated to

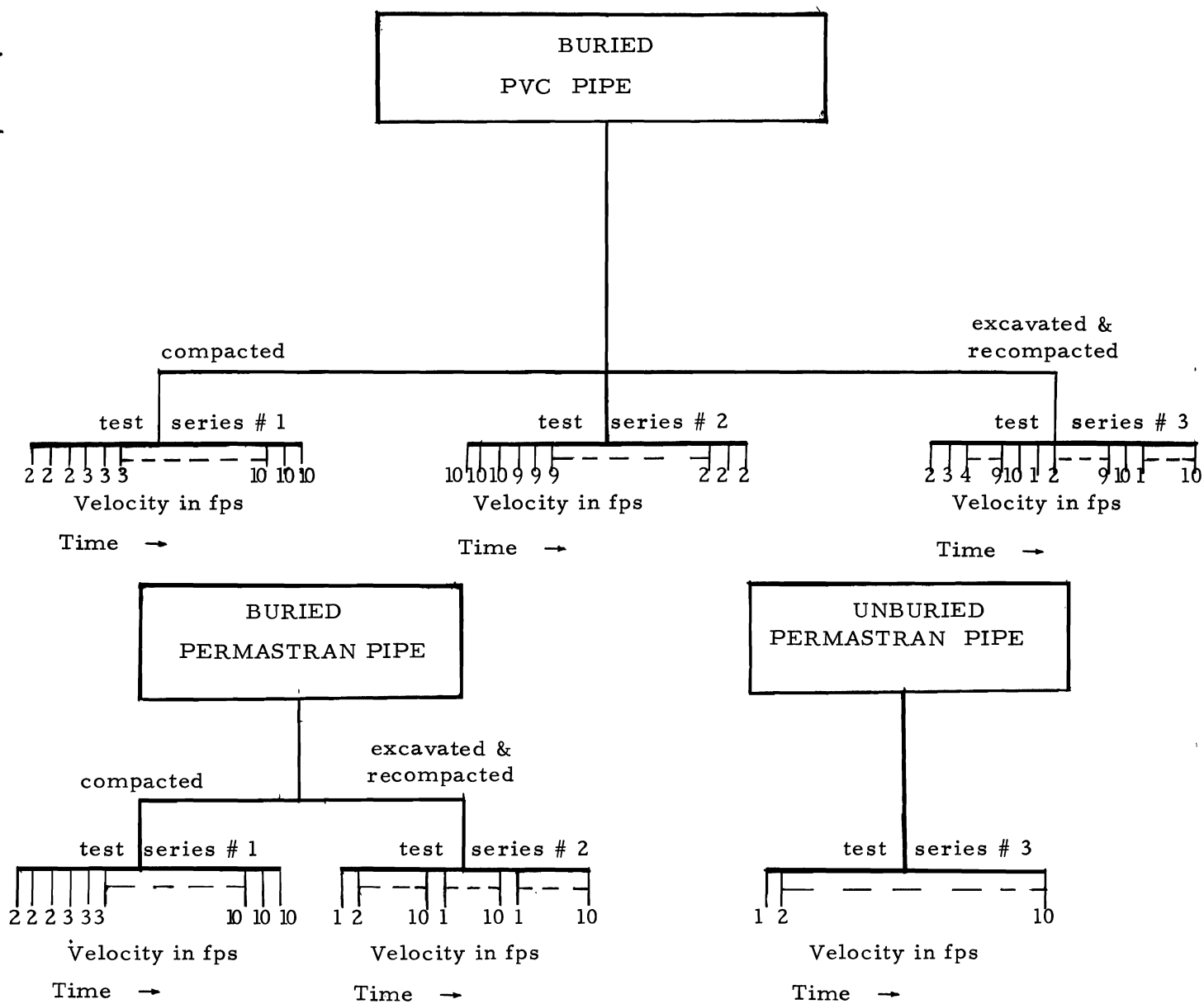


Fig. 5. Diagram showing sequence of water hammer test series.

the level of the bottom of the pipe and then again compacted around and over the pipe to a depth of approximately 1 1/2 feet above the pipe. The fill was not disturbed between the second and third series of tests on the PVC pipe. The first series of tests in each pipe replicated the measurements at each velocity three times before going to the next higher velocity. The last series of tests in each buried pipe also started with 1 or 2 fps, but increased the velocity by 1 fps increments for each succeeding test to 10 fps. These 10 tests were then replicated three times. The second series of tests in the PVC pipe started with 10 fps and replicated this three times before decreasing the velocity by 1 fps increments.

TEST RESULTS

The data obtained from the tests are summarized in Tables 2 through 7; for buried PVC pipe (Tables 2, 3, and 4), buried PERMASTRAN[®] pipe (Tables 5 and 6) and the unburied PERMASTRAN[®] pipe (Table 7). The velocities given in the second columns of these tables represent the initial velocities in the pipe prior to rapid valve closure. The velocities of the pressure waves are given in column 4, and were obtained by dividing the times in column 3 required for the wave to travel the distance between pressure transducers by the transducer spacing. This length equals 99.5 feet for the PVC pipe tests and 99.0 feet for the PERMASTRAN[®] pipe. The two pressure increments given for each test, referred to as the "normal pressure increment" and the "peak pressure increment," were obtained from the Visicorder charts as a judged mean high pressure for the former and the highest peak for the latter. In the case of the peak pressure for some of the tests, a much larger peak existed over a very small time interval in the order of .002 seconds. Since only a few of the tests exhibited this very short high pressure, it is believed to be caused by a mechanical

TABLE 2. FIRST SERIES OF TESTS ON PVC PIPE BURIED IN FILL

TEST NO.	V FPS	TIME SEC	WAVE VELOCITY (FT/SEC)		NORMAL PRESSURES(PSI)				MAXIMUM PRESSURES(PSI)				TIME FOR VALVE TO HAVE MOVED (SECONDS)		
					DOWNSTREAM		UPSTREAM		DOWNSTREAM		UPSTREAM				
			INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	1/2 IN	3 1/2 IN	4 IN
1	2	.080	124.0		32.2		26.6		36.0		26.6		.010	.050	.060
2	2	.080	124.0		37.9		35.5		44.5		36.4		.011	.063	.079
3	2	.075	132.5	12.70	37.9	36.0	35.5	32.5	44.5	41.6	36.4	33.1	.011	.053	.079
4	3	.080	124.0		53.0		51.5		58.7		50.6		.014	.058	.117
5	3	.080	124.0		51.1		47.9		56.8		47.9		.014	.058	.124
6	3	.080	124.0	12.40	51.1	51.7	47.9	49.1	56.8	57.4	47.9	48.8	.014	.058	.128
7	4	.081	122.5		68.1		59.5		69.1		59.5		.015	.056	.151
8	4	.082	121.0		62.5		63.9		68.1		63.9		.015	.056	.151
9	4	.082	121.0	12.15	65.3	65.3	65.7	63.0	70.0	69.1	65.7	63.0	.015	.057	.152
10	5	.082	121.0		73.7		79.7		75.5		79.7		.013	.053	.163
11	5	.082	121.0		75.5		74.8		77.4		74.8		.013	.056	.183
12	5	.082	121.0	12.10	73.7	74.3	72.9	75.8	77.4	76.8	72.9	75.8	.013	.055	.180
13	6	.082	121.0		85.0		69.2		86.9		69.2		.013	.055	.288
14	6	.082	121.0		83.1		67.3		86.9		69.2		.013	.056	.291
15	6	.083	119.5	12.05	88.0	85.4	71.1	69.2	88.8	87.5	71.1	69.8	.014	.056	.296
16	6	.083	119.5		91.4		79.3		96.3		79.3		.014	.056	.200
17	6	.082	121.0		90.7		65.4		94.4		65.4		.015	.052	.209
18	6	.082	121.0	12.05	92.5	91.5	69.2	71.3	94.4	95.1	69.2	71.3	.015	.052	.209
19	6	.084	118.0		90.7		69.2		94.4		69.2		.013	.053	.211
20	6	.083	119.5		88.8		69.2		94.4		71.1		.013	.053	.211
21	6	.083	119.5	11.90	88.8	89.4	72.9	70.4	94.4	94.4	72.9	71.1	.014	.054	.239
22	6	.083	119.5		88.8		71.1		94.4		71.1		.014	.054	.239
23	6	.084	118.0		91.4		87.9		94.4		87.9		.008	.050	.108
24	6	.083	119.5	11.90	92.5	90.9	95.4	84.8	98.2	95.7	95.4	84.8	.008	.050	.108
25	7	.083	119.5		106.5		99.1		111.4		99.1		.008	.046	.226
26	7	.084	118.0		107.7		99.1		111.4		101.0		.008	.046	.226
27	7	.084	118.0	11.85	107.7	107.3	102.1	100.1	113.3	112.1	102.8	101.0	.009	.046	.224
28	8	.084	118.0		119.0		99.5		122.8		99.5		.008	.044	.284
29	8	.085	117.0		122.8		105.8		128.4		105.8		.009	.039	.289
30	8	.084	118.0	11.75	122.8	121.5	108.5	104.6	126.5	125.9	108.5	104.6	.009	.042	.248
31	9	.085	117.0		130.3		130.9		134.1		130.9		.009	.041	.267
32	9	.085	117.0		136.0		125.3		139.8		125.3		.009	.042	.266
33	9	.084	118.0	11.75	111.4	125.9	123.4	126.5	116.3	130.1	123.4	126.5	.009	.042	.268
35	10	.084	118.0		119.0		125.3		122.8		125.3		.009	.042	.264
36	10	.084	118.0	11.80	119.0	119.0	115.9	120.6	122.8	122.8	115.9	120.6	.009	.045	.273

TABLE 3. SECOND SERIES OF TESTS ON PVC PIPE BURIED IN FILL

TEST NO.	V FPS	TIME SEC	WAVE VELOCITY (FT/SEC)		NORMAL PRESSURES (PSI)				MAXIMUM PRESSURES (PSI)				TIME FOR VALVE TO HAVE MOVED (SECONDS)		
					DOWNSTREAM		UPSTREAM		DOWNSTREAM		UPSTREAM				
			INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	1/2 IN	3 1/2 IN	4 IN
1	10	.084	1180		155.6		140.2		146.6		125.3		.006	.046	.141
2	10	.084	1180		153.3		157.1		157.8		157.1		.009	.044	.103
3	10	.084	1180	1180	153.3	154.1	149.6	149.0	157.8	154.1	149.6	144.0	.009	.041	.101
4	9	.084	1180		137.6		130.9		142.1		130.9		.008	.040	.099
5	9	.084	1180		139.8		129.0		142.1		129.0		.007	.042	.099
5	9	.084	1180		135.3		129.0		142.1		129.0		.010	.042	.268
6	9	.084	1180	1180	139.8	138.1	136.5	131.4	142.1	142.1	136.5	131.4	.008	.041	.092
7	8	.084	1180		115.0		119.7		135.3		119.7		.008	.040	.085
8	8	.084	1180		124.0		127.2		126.3		127.2		.008	.042	.087
9	8	.085	1170	1175	124.0	121.0	123.4	123.4	126.3	129.3	123.4	123.4	.008	.042	.085
10	7	.084	1180		106.0		112.2		108.2		112.2		.007	.039	.082
11	7	.084	1180		103.7		114.1		108.2		114.1		.007	.040	.080
12	7	.084	1180	1180	103.7	104.5	114.1	113.4	108.2	108.2	114.1	113.4	.007	.040	.080
13	6	.084	1180		90.2		99.1		92.5		99.1		.007	.040	.080
14	6	.084	1180		87.9		99.1		92.5		99.1		.007	.039	.080
15	6	.085	1170	1175	87.9	88.7	97.2	98.5	92.5	92.5	97.2	98.5	.007	.039	.081
16	5	.084	1180		74.4		80.4		76.7		80.4		.007	.038	.075
17	5	.084	1180		72.2		81.5		76.7		81.5		.007	.039	.075
18	5	.084	1180	1180	72.2	72.9	81.5	81.2	74.4	75.9	81.5	81.2	.007	.039	.077
19	4	.083	1195		56.4		64.3		58.6		64.3		.007	.039	.075
20	4	.083	1195		56.4		64.3		60.9		64.3		.007	.039	.077
21	4	.084	1180	1190	56.4	56.4	62.8	63.8	58.6	59.4	62.8	63.8	.007	.040	.077
22	3	.083	1195		45.1		43.0		49.6		43.0		.007	.040	.054
23	3	.084	1180		45.1		46.7		49.6		46.7		.007	.039	.075
24	3	.083	1195	1190	44.2	44.8	47.5	45.8	47.4	48.9	47.5	45.8	.007	.041	.080
25	2	.083	1195		28.4		32.9		38.3		32.9		.007	.038	.048
26	2	.083	1195		28.4		33.7		38.3		33.7		.004	.036	.046
27	2	.083	1195	1195	28.4	28.4	28.8	31.8	31.6	36.1	28.8	31.8	.007	.038	.048

TABLE 4. THIRD SERIES OF TESTS ON PVC PIPE BURIED IN FILL

TEST NO.	V FPS	TIME SEC	WAVE VELOCITY (FT/SEC) INDIVID.	NORMAL PRESSURES (PSI)		MAXIMUM PRESSURES (PSI)		TIME FOR VALVE TO HAVE MOVED (SECONDS)		
				DOWNSTREAM INDIVID.	UPSTREAM INDIVID.	DOWNSTREAM INDIVID.	UPSTREAM INDIVID.	1/2 IN	3 1/2 IN	4 IN
2	2	.082	1210	21.0	30.3	25.5	30.3	.006	.043	.087
3	3	.082	1210	41.1	43.0	45.6	43.0	.008	.052	.107
4	4	.081	1225	63.9	59.8	68.4	59.8	.007	.048	.145
5	5	.081	1225	79.8	67.3	84.4	67.3	.006	.046	.140
6	6	.081	1225	95.8	93.5	100.4	93.5	.007	.048	.132
7	7	.081	1225	107.2	115.9	111.8	115.9	.008	.048	.130
8	8	.080	1240	125.5	123.4	130.0	123.4	.008	.049	.134
9	9	.081	1225	136.9	145.9	141.4	145.9	.008	.049	.130
10	10	.081	1225	152.8	155.2	155.1	155.2	.008	.048	.136
11	1	.081	1225	16.9	16.1	18.2	16.5	.008	.047	.096
12	2	.080	1240	21.4	31.0	25.5	31.0	.008	.048	.096
13	3	.081	1225	43.3	54.2	52.5	54.2	.007	.046	.095
14	4	.080	1240	44.3	56.1	53.4	56.1	.007	.045	.093
15	5	.081	1225	70.7	83.4	79.8	83.4	.007	.045	.096
16	6	.081	1225	89.0	99.1	100.4	99.1	.008	.046	.099
17	7	.081	1225	102.6	117.8	111.8	117.8	.007	.046	.120
18	8	.082	1210	118.6	134.6	127.7	134.6	.007	.046	.122
19	9	.082	1210	136.9	152.6	141.4	152.6	.007	.044	.121
20	10	.082	1210	152.8	162.7	159.7	162.7	.008	.046	.123
21	1	.082	1210	11.4	15.0	16.0	15.0	.009	.049	.097
22	2	.080	1240	20.5	29.9	27.4	29.9	.008	.047	.095
23	3	.081	1225	37.4	48.6	43.3	48.6	.007	.046	.095
24	4	.081	1225	57.0	67.3	66.1	67.3	.008	.047	.097
25	5	.081	1225	70.7	82.3	79.8	82.3	.008	.048	.098
26	6	.083	1195	86.7	101.0	95.8	101.0	.008	.048	.121
27	7	.083	1195	104.9	117.8	109.5	117.8	.009	.049	.124
28	8	.082	1210	118.6	134.6	127.7	134.6	.008	.048	.123
29	9	.082	1210	139.1	152.6	141.4	152.6	.007	.045	.122
30	9	.084	1180	134.6	123.4	139.1	123.4	.008	.048	.301
31	10	.083	1195	157.4	140.2	162.0	140.2	.006	.047	.176
32	10	.082	1210	152.8	168.3	157.4	168.3	.006	.045	.125
33	10	.084	1180	143.7	125.3	148.3	125.3	.006	.046	.304

TABLE 5. FIRST SERIES OF TESTS ON PERMASTRAN PIPE BURIED

TEST NO.	V FPS	TIME SEC.	WAVE VELOCITY (FT/SEC)		NORMAL PRESSURES(PSI)				MAXIMUM PRESSURES(PSI)				POSITION OF VALVE AT (INCHES)				
					DOWNSTREAM		UPSTREAM		DOWNSTREAM		UPSTREAM						
			INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	INDIVID.	AVE.	.01 SEC	.02 SEC	.03 SEC	.04 SEC	.05 SEC
1	2	.065	1520		45.0		39.0		48.3		39.2		3.2	3.9	4.8	5.2	6.0
2	2	.065	1520		46.7		41.7		54.3		42.6		2.8	3.6	4.3	5.2	6.0
3	2	.063	1570	15.35	45.0	45.6	41.7	40.8	54.3	52.3	43.0	41.6	2.9	3.9	4.7	5.6	6.0
4	3	.064	1545		60.0		56.8		73.3		58.6		3.0	3.9	4.7	5.5	6.0
5	3	.063	1570		71.7		64.8		84.7		68.0		3.1	3.9	4.7	5.5	6.0
6	3	.062	1595	15.70	63.3	65.0	56.8	59.5	73.0	77.0	58.8	61.8	3.3	4.0	4.9	5.9	6.0
7	4	.062	1595		76.7		73.7		96.0		74.5		3.0	3.9	4.7	6.0	6.0
8	4	.066	1495		80.0		61.7		100.3		74.8		3.1	4.0	4.8	5.8	6.0
9	4	.065	1520	15.35	81.7	79.4	72.9	69.4	102.7	99.7	76.7	75.3	2.9	3.7	4.2	5.4	6.0
10	5	.066	1495		101.7		86.0		101.7		87.1		3.0	3.9	4.8	5.8	6.0
11	5	.066	1495		101.7		86.0		106.7		89.8		3.1	3.9	4.7	5.6	6.0
12	5	.065	1520	15.05	106.7	103.3	101.4	91.1	129.3	112.5	103.2	93.4	3.1	3.9	4.6	5.7	6.0
13	6	.065	1520		129.6		119.7		145.4		122.7		3.0	3.8	4.7	5.5	6.0
14	6	.065	1520		129.3		121.5		146.9		132.0		3.3	4.0	4.8	6.0	6.0
15	6	.065	1520	15.20	130.4	129.8	122.7	121.3	146.9	146.4	123.8	126.2	3.3	4.1	5.1	6.0	6.0
16	7	.064	1545		146.0		144.0		147.8		145.9		3.1	3.9	4.7	5.9	6.0
17	7	.063	1570		146.0		144.0		147.5		145.9		3.1	3.8	4.6	5.8	6.0
18	7	.062	1595	15.70	145.7	145.9	144.0	144.0	147.5	147.6	145.9	145.9	3.1	4.0	4.8	5.9	6.0
19	8	.063	1570		166.3		166.4		167.8		166.4		3.1	3.9	4.7	5.6	6.0
20	8	.062	1595		166.0		166.4		167.8		166.4		3.0	3.8	4.7	5.6	6.0
21	8	.063	1570	15.75	167.2	166.5	166.4	166.4	168.7	168.1	166.4	166.4	3.1	3.8	4.5	5.4	6.0
22	9	.062	1595				183.3						3.1	3.9	4.6	5.6	6.0
23	9	.061	1620		185.1		187.0		186.3		188.9		3.1	3.9	4.8	5.6	6.0
24	9	.061	1520	16.10	183.3	184.2	185.1	277.7	185.1	185.7	187.0	187.9	3.1	3.8	4.6	5.4	6.0

TABLE 6. SECOND SERIES OF TEST ON PERMASTRAN PIPE BURIED

TEST NO.	V FPS	TIME SEC.	WAVE VELOCITY (FT/SEC) INDIVID.	NORMAL PRESSURES (PSI)		MAXIMUM PRESSURES (PSI)		POSITION OF VALVE AT (INCHES)				
				DOWNSTREAM	UPSTREAM	DOWNSTREAM	UPSTREAM					
				INDIVID.	INDIVID.	INDIVID.	INDIVID.	.01 SEC	.02 SEC	.03 SEC	.04 SEC	.05 SEC
1	1	.064	1545	20.0	15.7	20.7	16.6	3.2	3.5	4.2	5.1	6.0
2	2	.065	1520	40.7	38.5	46.7	38.8	3.2	4.1	4.9	5.7	6.0
3	3	.063	1570	57.0	54.6	69.3	57.0	3.2	3.9	4.7	5.5	6.0
4	4	.065	1520	80.7	77.3	100.0	77.3	3.2	4.0	4.9	5.7	6.0
5	5	.063	1570	106.0	103.4	125.7	104.2	3.3	4.2	5.1	5.7	6.0
6	6	.063	1570	127.8	127.4	152.2	127.8	3.3	3.9	4.7	5.6	6.0
7	7	.063	1570	146.6	139.0	147.2	150.2	3.4	4.2	5.1	5.8	6.0
8	8	.063	1570	167.2	167.9	168.4	169.1	3.4	4.2	5.0	5.8	6.0
9	9	.063	1570	187.2	179.5	191.0	181.4	3.4	4.2	5.0	5.8	6.0
10	10	.064	1545		204.6		207.7	3.4	4.1	4.9	5.7	6.0
11	1	.063	1570	20.0	16.6	21.7	18.5	3.1	3.9	4.7	5.7	6.0
12	2	.063	1570	40.7	38.8	50.0	39.8	3.3	4.2	5.1	6.0	6.0
13	3	.063	1570	55.7	54.6	63.3	56.4	3.3	4.0	4.8	6.0	6.0
14	4	.063	1570	80.3	81.4	101.7	83.2	3.7	4.2	5.0	6.0	6.0
15	5	.065	1520	105.0	101.9	126.7	102.3	3.3	4.1	5.0	5.9	6.0
16	6	.064	1545	127.5	124.7	148.4	127.4	3.3	4.1	5.1	6.0	6.0
17	7	.064	1545	143.6	140.9	143.9	141.3	3.5	4.3	5.3	6.0	6.0
18	8	.064	1545	166.6	166.0	167.5	166.4	3.3	4.2	5.0	6.0	6.0
19	9	.064	1545	186.6	179.5	187.2	181.4	3.4	4.2	5.0	6.0	6.0
20	10	.063	1570	202.1	196.9	206.6	198.0	3.3	4.2	5.2	5.7	6.0
21	1	.064	1545	20.0	16.6	21.7	17.0	3.3	4.3	5.2	6.0	6.0
22	2	.063	1570	43.3	42.9	55.0	44.0	3.4	4.3	5.2	6.0	6.0
23	3	.062	1595	56.7	57.3	65.0	59.0	3.3	4.3	5.1	5.9	6.0
24	4	.063	1570	78.3	81.0	98.3	82.0	3.3	4.3	5.1	6.0	6.0
25	5	.063	1570	104.0	101.1	123.3	102.3	3.4	4.1	4.8	6.0	6.0
26	6	.063	1570	121.5	119.7	143.3	120.0	3.5	4.1	5.0	5.8	6.0
27	7	.063	1570	144.5	142.8	146.9	144.7	3.5	4.1	5.0	5.8	6.0
28	8	.063	1570	160.3	162.1	162.1	162.5	3.3	3.9	5.0	5.8	6.0
29	9	.063	1570	182.7	180.6	184.8	185.3	3.3	3.9	5.1	6.0	6.0
30	10	.063	1570	205.1	198.8	206.6	200.7	3.3	4.0	4.7	5.7	6.0

TABLE 7. TEST RESULTS FROM PERMASTRON PIPE IN UNBURIED CONDITION

TEST NO.	V FPS	TIME SEC.	WAVE VELOCITY (FT/SEC) INDIVID.	NORMAL PRESSURES (PSI)		MAXIMUM PRESSURES (PSI)		POSITION OF VALVE AT (INCHES)				
				DOWNSTREAM INDIVID.	UPSTREAM INDIVID.	DOWNSTREAM INDIVID.	UPSTREAM INDIVID.	.01 SEC	.02 SEC	.03 SEC	.04 SEC	.05 SEC
1	1	.065	1520	20.0	.0	21.7	20.0	2.8	3.5	4.2	5.1	5.7
2	2	.065	1520	40.0	39.8	41.7	39.8	3.0	3.6	4.3	5.2	5.9
3	3	.065	1520	58.3	56.4	61.7	56.4	3.0	3.6	4.3	5.2	5.9
4	4	.065	1520	78.3	76.2	81.7	78.6	3.7	4.1	4.8	5.9	6.0
5	5	.064	1545	101.7	92.5	103.3	92.5	3.1	4.0	4.9	5.8	5.9
6	6	.068	1455	123.0	123.5	124.5	123.5	3.2	4.0	4.8	5.7	6.0
7	7	.067	1475	140.9	139.0	143.9	139.0	3.0	3.9	4.7	5.4	5.8
8	8	.067	1475	161.8	152.5	166.3	152.5	3.1	3.9	4.6	5.4	5.7
9	9	.067	1475	181.2	163.3	184.2	163.3	2.6	3.6	4.3	5.1	5.8
10	10	.067	1475	199.1	167.9	202.1	167.9	2.7	3.6	4.4	5.2	5.7

vibration of a pipe support. These very short severe pressures are included in the peak pressures of those tests in which they occurred and can be identified by noting the substantial difference from the normal pressure.

In the process of obtaining the first series of test results, a number of additional runs were obtained at $V_o = 6$ fps in attempting, with little improvement, to achieve faster valve closure. After completing this first series of tests the valve was outfitted with grease fittings. Lubricating the seats of the valve did decrease the closure times substantially for subsequent tests.

It should be noted that a substantial reduction in pressure occurred between tests No. 32 and 33 of this first series (Table 2). It is not possible to determine absolutely the cause of this reduction, but it is believed to be associated with the valve not closing fast enough over the last 1/2 inch of its travel. While the total closure time is essentially the same for these two tests, the drop in pressure could be the result of the valve "hanging-up" at the top of the last 1/2 inch in test No. 33, whereas in test No. 32 this "hang-up" occurred nearer the end of the last 1/2 inch of travel. It should also be noted that the pressures in this first series with $V_o = 10$ fps are too low and should be disregarded. Also during the second test series on the PVC pipe with $V_o = 10$ fps the 8-inch steel pipe upstream from the test section split open at one of the welded joints at an elbow, thus preventing the third replication with $V_o = 10$ from being obtained.

The fill material was removed and compacted again around and above the pipe between the second and third series of tests (between the time the data in Tables 3 and 4 were obtained) on the PVC pipe. The pipe and transducer failure during the final test of the third series prevented a planned fourth series of tests to be conducted in which the velocities V_o were to begin with 10 fps and decrease in 1 fps increments.

The fill was also excavated and compacted again around and over the PERMASTRAN® pipe between the first and second series of tests using this pipe.

Measurements on the PVC pipe indicate the average inside diameter equals 6.09 inches and that its wall thickness averages 0.276 inches. The PERMASTRAN® pipe has an average inside diameter equal to 6.25 inches and a wall thickness equal to 0.168 inches. In the case of the PERMASTRAN® pipe, the same manometer readings were set to establish the indicated integer values of velocities in Tables 5 through 7 as were set for the PVC pipe. These settings assumed that its inside diameter equals 6.09 inches. Consequently, the actual velocities in the PERMASTRAN® pipe are equal to the indicated amount multiplied by $(6.09/6.25)^2 = .95$. In all of the analyses or computations in these analyses which are given later, the corrected velocities have been used.

Pressure Wave Velocities

In order to compute wave velocities, the modulus of elasticity and Poisson's ratio for the pipe material must be known. These values were supplied by Johns-Manville and are:

$$\begin{array}{ll} \text{Modulus of elasticity, } E = 500,000 \text{ psi} \\ \text{Poisson's ratio, } \mu = 0.46 \end{array}$$

for the PVC pipe and

$$\begin{array}{ll} \text{Modulus of elasticity, } E = 1,440,000 \text{ psi} \\ \text{Poisson's ratio, } \mu = 0.52 \end{array}$$

for the PERMASTRAN® pipe.

The value for Poisson's ratio for the PERMASTRAN® pipe, $\mu = 0.52$, is larger than the theoretical maximum value of 0.5 for an isotropic material of which PERMASTRAN® is not. The .52 value may well be due to the fact that the glass fiber which is wrapped around polyvinylchloride core in the manufacture of the PERMASTRAN® pipe occurs on the bias in two opposite directions, or to the method used to

determine Poisson's ratio.

It should be pointed out that the development of Eq. 1 assumes the pipe wall consists of an isotropic material with a stress-strain diagram defined by the modulus of elasticity and, therefore, the effects in Eq. 1 from the expansion of the pipe wall may not be adequate in describing water hammer velocities in the PERMASTRAN® pipe.

Using the above values for E and μ , the theoretical wave velocities as computed by Eq. 1 using c_1 for the pipes anchored throughout against axial movement are:

$$(a) \text{ PVC pipe - thin walled } (c_1 = 1 - (.46)^2 = .788)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{5 \times 10^5} \right) \left(\frac{6.09}{0.276} \right) (.788)}} = 1,395 \text{ fps}$$

$$(b) \text{ PVC pipe - thick walled } \left(c_1 = \frac{2(.276)}{6.09} (1 + .46) + \frac{6.09 (1 - (.46)^2)}{6.09 + .276} = .887 \right)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{5 \times 10^5} \right) \left(\frac{6.09}{0.276} \right) (.887)}} = 1,322 \text{ fps}$$

$$(c) \text{ PERMASTRAN® pipe - thin walled } (c_1 = 1 - (.52)^2 = .730)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{14.4 \times 10^5} \right) \left(\frac{6.25}{.168} \right) (.730)}} = 1,830 \text{ fps}$$

$$(d) \text{ PERMASTRAN}^{\circledR} \text{ pipe - thick walled } \left(c_1 = \frac{2(.168)}{6.25} (1 + .52) + \frac{6.25(1 - (.52)^2)}{6.25 + .186} = .792 \right)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{14.4 \times 10^5} \right) \left(\frac{6.25}{.168} \right) (.792)}} = 1,766 \text{ fps}$$

The wave velocities for the pipes computed under the assumption that the ends are fixed but the pipe contains expansion joints throughout are:

$$(a) \text{ PVC pipe - thin walled } (c_1 = 1)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{5 \times 10^5} \right) \left(\frac{6.09}{0.276} \right)}} = 1,250 \text{ fps}$$

$$(b) \text{ PVC pipe - thick walled } \left(c_1 = \frac{2(.276)}{6.09} (1 + .46) + \frac{6.09}{6.09 + .276} = 1.09 \right)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{5 \times 10^5} \right) \left(\frac{6.09}{.276} \right) (1.09)}} = 1,202 \text{ fps}$$

$$(c) \text{ PERMASTRAN}^{\circledR} \text{ pipe - thin walled } (c_1 = 1)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{14.4 \times 10^5} \right) \left(\frac{6.25}{.168} \right)}} = 1,595 \text{ fps}$$

$$(d) \text{ PERMASTRAN}^{\circledR} \text{ pipe - thick walled } \left(c_1 = \frac{2(.168)}{6.25} (1 + .52) + \frac{6.25}{6.25 + .168} = 1.056 \right)$$

$$a = \frac{12 \sqrt{3 \times 10^5 / 1.94}}{\sqrt{1 + \left(\frac{3 \times 10^5}{14.4 \times 10^5} \right) \left(\frac{6.25}{.168} \right) (1.056)}} = 1,557 \text{ fps}$$

While the diameter-to-wall thickness ratio is close to the limit for thick-walled pipe in the case of the PVC pipe, clearly only the thin-walled velocities apply, since in each case the computed thick-walled wave velocities are less than the thin-walled values.

The pressure wave velocity in the unburied 6-inch PVC pipe, as determined from the previous tests (see Watters, 1971, Table 4), was 1120 fps. The average wave velocity from the first test series equals 1210 fps, for the second series equals 1185 fps, and for the third series after the fill was compacted again equals 1215 fps. Furthermore, the individual wave velocities in Tables 2, 3, and 4 show a slight decreasing trend with the number of the test. This trend is most pronounced in Table 2, in which the average of the first three tests (1270 fps) is 7.3 percent greater than the average of the final three tests of this series (1180 fps). This slight decrease in wave velocity with number of water hammer waves which have preceded it in the pipe, represents the decrease in support that the pipe receives from the surrounding soil. It appears, however, that the trend is small enough to be of very little importance.

The difference between the wave velocities in the unburied and buried pipes represents the effects of the soil support. In the case of the 6-inch PVC pipe this resistance to motion offered by the soil has

increased the wave velocity on the average of 7 percent (from 1120 fps to 1210 fps), and when first compacted 12 percent.

The increase in velocity in buried over unburied PVC pipe is less than the difference between the velocity computed by Eq. 1 for no axial movement (1390 fps) and that computed for expansion joints throughout (1250 fps). Consequently, it could be concluded that the major effect from the compacted fill is to prevent axial movement of the pipe. Clearly, the unburied pipe with an expansion joint which does not slip without resistance, would be expected to give wave velocities between the computed values with and without axial restraint.

In the case of the more rigid PERMASTRAN® pipe, the effects on the wave velocity of burying the pipe are even smaller (see Tables 5, 6, and 7). No decreasing wave velocity trend can be detected with the number of the test in PERMASTRAN® pipe, and the average velocity in the buried pipe is 3.5 percent greater than in the unburied pipe, a difference which is of about the same order of magnitude to which the velocity can be determined from the Visicorder charts. The conclusion, therefore, is that burying PERMASTRAN® pipe in a well compacted fill has a minor effect on water hammer.

Of greater significance in the case of the PERMASTRAN® pipe is the difference between the theoretical velocity computed from Eq. 1 and the measured velocities. If the value for the axially restrained thin-walled pipe is used (1830 fps), the average measured velocity (1555 fps) is 15 percent less than the computed. Entrained air in the water would explain much of this difference. For example, the entrained air effect might be assumed equal to the difference between the computed and average measured velocity in the PVC pipe (1395 fps - 1200 fps = 195 fps). This assumed difference of 195 fps in the wave velocities due to entrained air is realistic. Using the theory presented by Olsen (1966) in Problem 2-60, shows that approximately .06 percent (volume basis) of air in water could reduce the wave velocity by this amount. The remaining difference

between the theoretical and measured wave velocities in the PERMASTRAN® pipe is then approximately 80 fps. A reduction of Poisson's ratio from the given 0.52 to 0.46 would account for 60 fps of this 80 fps.

Magnitude of Pressure Increases

The development of Eqs. 1 and 2 assumes, among other things, that the valve is closed during zero time. Due to physical limitations water hammer waves always result from a time dependent closure of a valve, but often the time of closure is rapid enough that the effect, for all practical purposes, is identical to a zero time closure. As mentioned earlier in the report, the first tests were conducted when the complete closure of the valve occurred over too long a time increment, particularly at the higher velocities. Much of the difference between the classical square water hammer wave pattern and the actual shape observed from the Visicorder charts is due to the time dependent closure of the valve.

A computer program was developed to solve the water hammer problem resulting from a programmed valve closure. A brief description of the method of solution used in this program, as well as the means for denoting the valve closure relationship, is given in Appendix A. The solution obtained from this program under the assumption that the valve closed very rapidly (i. e., within .00001 seconds) with an initial velocity $V_o = 10$ fps is plotted as the solid lines in Fig. 6. The slight difference from the vertical shown by these lines is not actual, but due to the time interval Δt used in the solution (which, for this solution, is .0083 seconds). The solution shows the complete change in pressure within this time interval, and the plotting of the pressure curves simply connected pressure at consecutive time intervals by straight lines. A better plotting would have constructed vertical lines midway between the time intervals at which the pressure changed. This solution gives an increase in pressure to 212 psi from 50 psi or 162 psi, which corresponds exactly with that given by Eq. 3a.

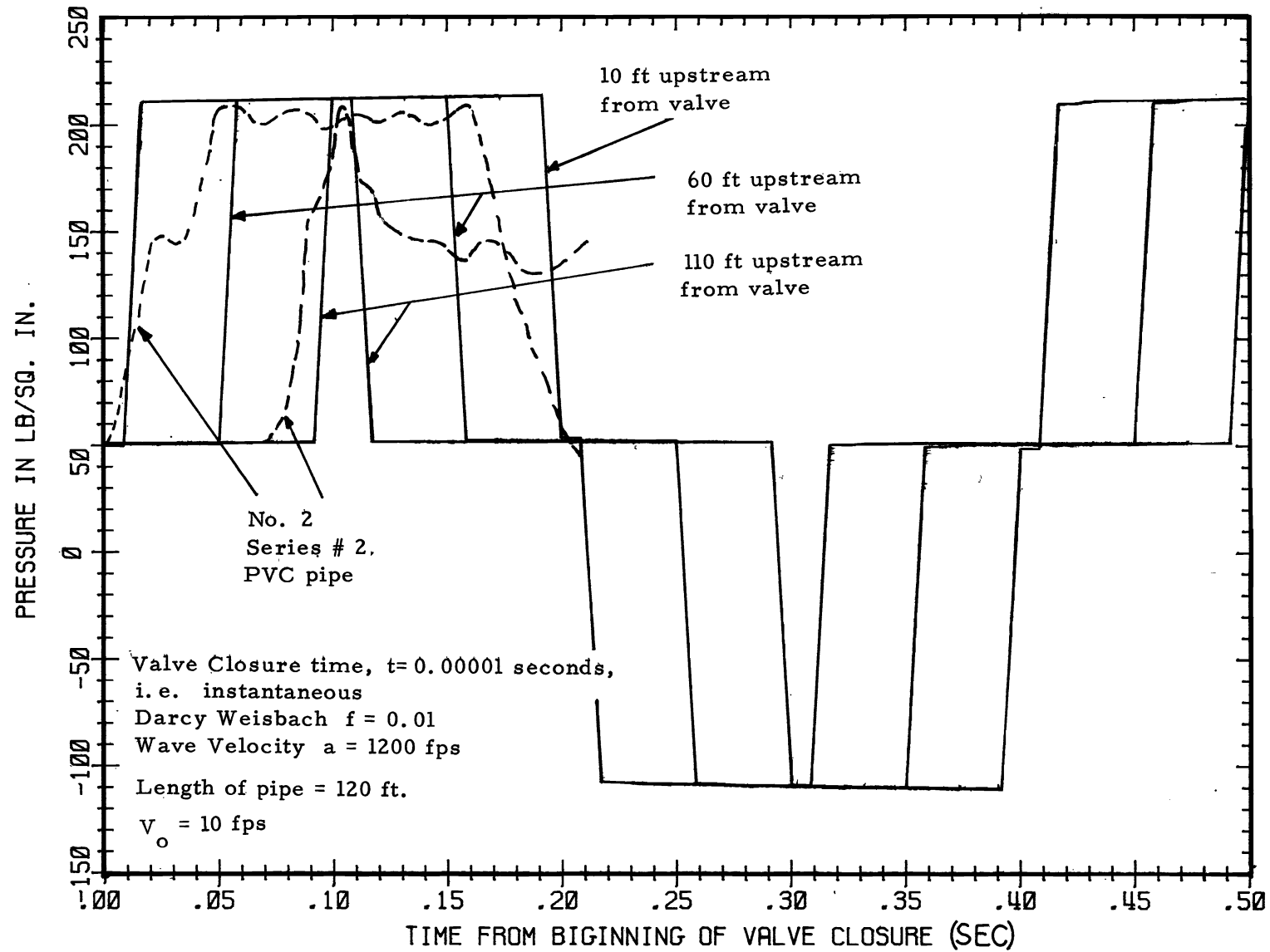


FIG. 6. WATER HAMMER PRESSURE WAVE RESULTING FROM PROGRAMMED VALVE CLOSURE

If, however, the valve closure occurs during some finite time, the pressure wave will not be square, and if the time is of sufficient duration for the reflected wave to arrive back at the valve during its closure, the maximum pressure will be reduced. To illustrate this, the results from a computer solution with $a = 1200$ fps and $V_o = 10$ fps as in the previous solution, but with a programmed valve closure, are given in Fig. 7. The time dependency of the valve closure is shown by the table within the graph. Perhaps a more representative description of the valve closure for this problem is given by the time variation of the dimensionless value C_4 , defined by Eq. A-13 (in Appendix A), and which has been plotted in Fig. 8. The test results from test No. 2, series #2 on PVC pipe are superimposed on both the solutions given in Figs. 6 and 7, as dashed lines. The different position of the experimental and theoretical lines on Fig. 7 for the drop in pressure is associated with the fact that the theoretical solution specified a pipe length of 140 feet. If the duration of the positive pressure is disregarded, clearly closer agreement exists between the experimental results and the theoretical solution with the programmed valve closure than with the instantaneous valve closure. Good agreement between the theoretical and experimental curves is not expected over the negative wave cycle. Any column separation would cause the negative cycle to be longer. The computer solution assumes a reservoir exists upstream, whereas the experimental set-up goes to an 8-inch pipe preceded by elbows, valves, and finally the pump. Other explanations for differences are that the computer solution does not account in any way for the effects of the compacted soil, if any exist. Furthermore, the discharge coefficients C_d given for the valve for the various partially closed positions are only rough estimates. Clearly as the valve becomes nearly closed C_d should be smaller, but no data are available (nor were any tests conducted) to determine the correct values of C_d for different positions of the gate in the valve. In addition, the area through which water can flow between the gate of the valve and its two seats is not

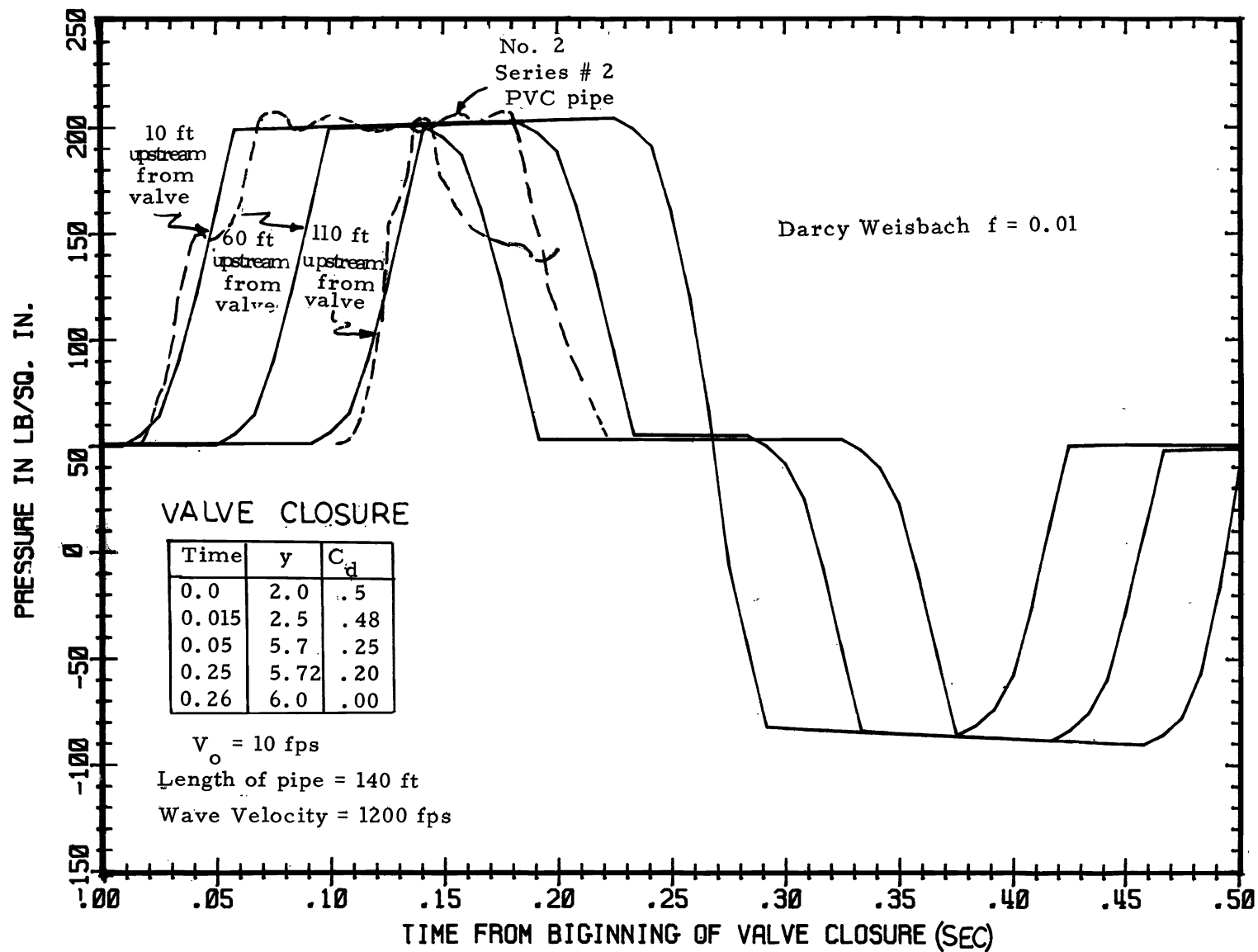


FIG. 7. WATER HAMMER PRESSURE WAVE RESULTING FROM PROGRAMMED VALVE CLOSURE

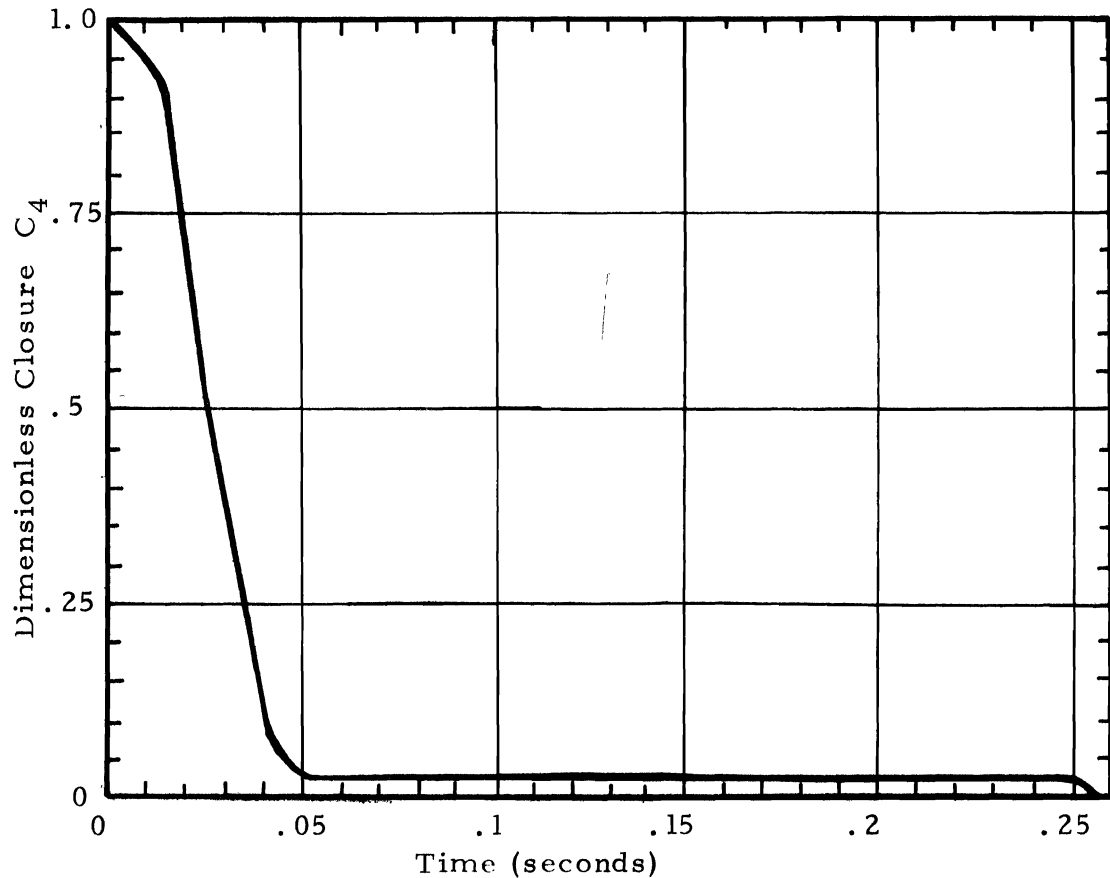


Fig. 8. Valve closure characteristics used in computer solution of Figs. 7, 9, and 10.

described completely in the computations in the computer program. Rather than attempting to obtain complete agreement between the computer solution and the measured pressure waves, the intent is to demonstrate that measured pressure patterns are, in fact, in good agreement with water hammer theory, providing this theory adequately accounts for the physical influences.

The results from a computer solution with the same programmed valve closure as was used to obtain the solution in Fig. 7, but for which the pipe is 20 feet shorter, is shown in Fig. 9. In this solution, only a 10-foot length of pipe exists between the upstream pressure-time wave which is shown and the reservoir. Note that the peak pressure at this point is considerably below the peak pressures further downstream. For

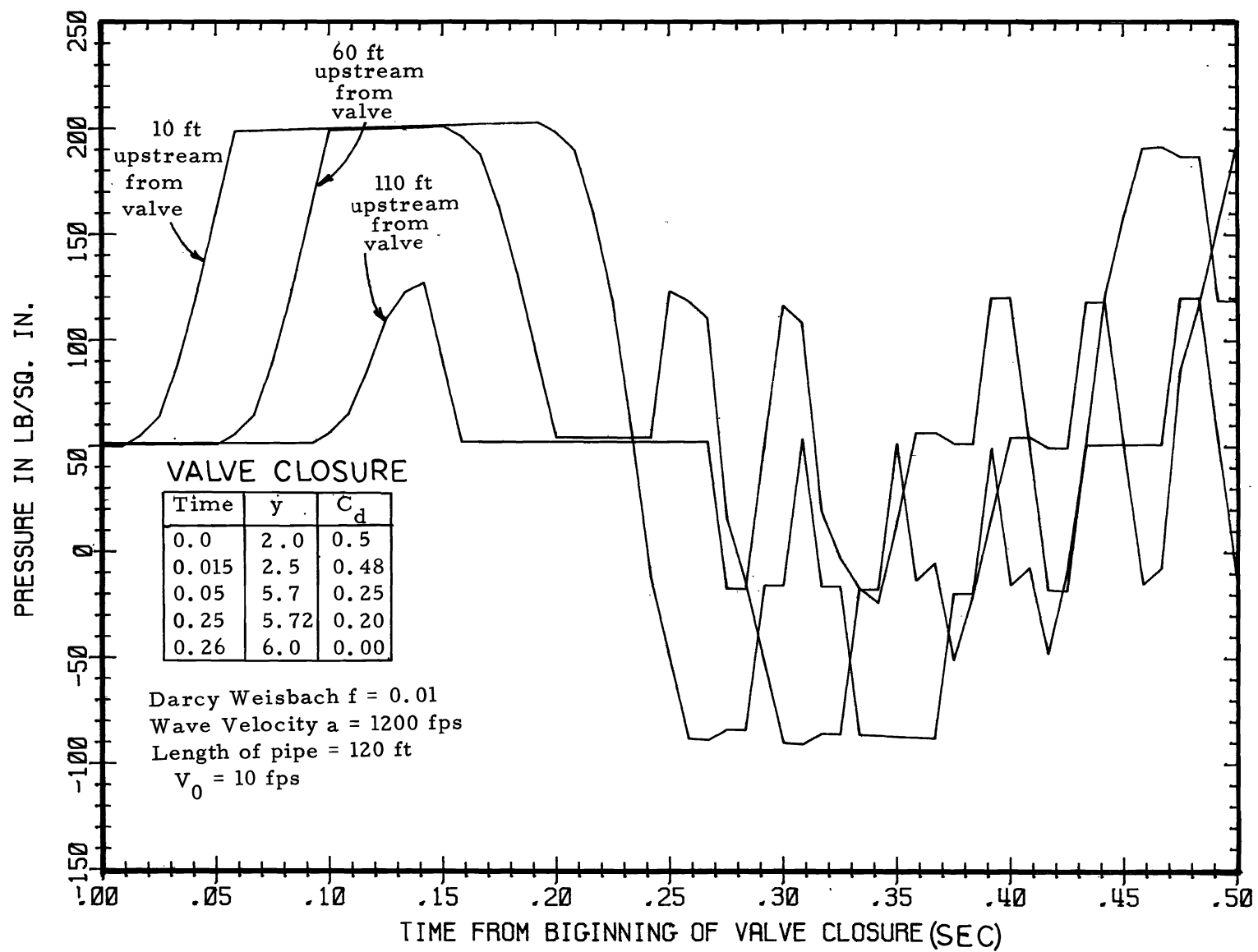


FIG. 9. WATER HAMMER PRESSURE WAVE RESULTING FROM PROGRAMMED VALVE CLOSURE

an instant valve closure this peak pressure would equal the downstream peak pressure. Also in this solution the effects of completing closure after the reflected wave has returned causes a complex pattern of waves to occur after the initial positive wave.

Fig. 10 gives the results from a computer solution in which the velocity in the pipe was specified as $V_o = 6$ fps, but valve closure characteristics remain as in the solution in Fig. 7. The maximum pressure from this solution also agrees closely with the measured pressures in the tests for which $V_o = 6$ fps in the PVC pipe.

To provide an overview concerning the agreement of the measured and theoretical pressure values, Figs. 11 through 18 have been prepared. The first four of these figures (11-14) show the increases in pressures due to the water hammer as computed by Eq. 3 using the actual measured velocity from each test against the measured pressures from the test using the data obtained from tests on PVC pipe. The different plotting symbols distinguish the test series from which the data were obtained. In most cases the upstream normal and peak pressures were recorded as being identical. The last four of these figures (15-18) consist of the similar plots constructed using the data from the tests on the PERMASTRAN[®] pipe.

CONCLUSIONS

In summary, the experimental test results of water hammer pressure waves in both PVC and PERMASTRAN[®] pipe are in reasonable agreement with those predicted by commonly used equations. In both types of pipe the velocity of the pressure wave is less than that predicted by theory. The difference is easily accounted for in PVC pipe by entrained air, and for PERMASTRAN[®] pipe, in which the difference is greater, by perhaps a better definition of the material's Poisson's ratio and modulus of elasticity. The measured pressure increases due

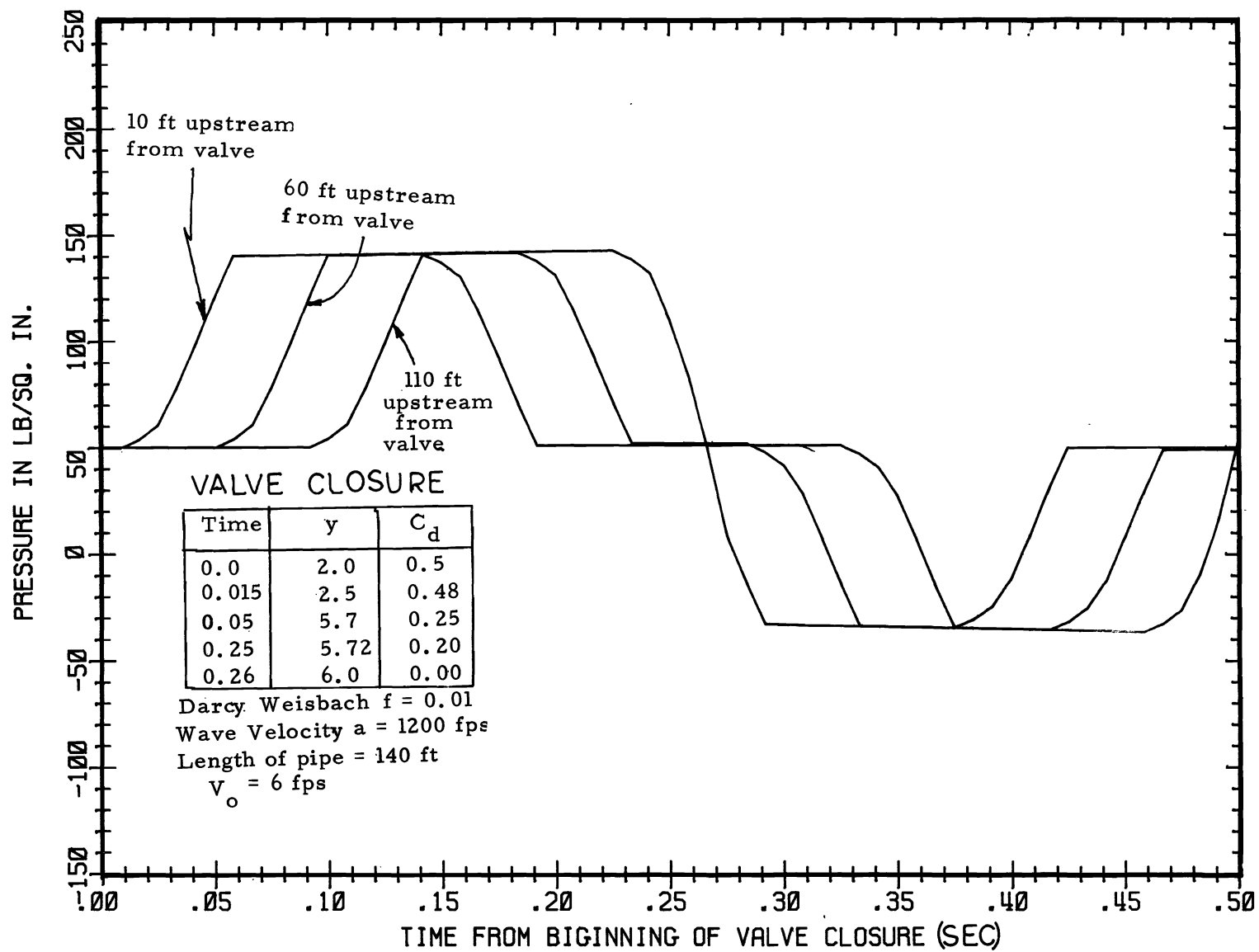


FIG. 10. WATER HAMMER PRESSURE WAVE RESULTING FROM PROGRAMMED VALVE CLOSURE

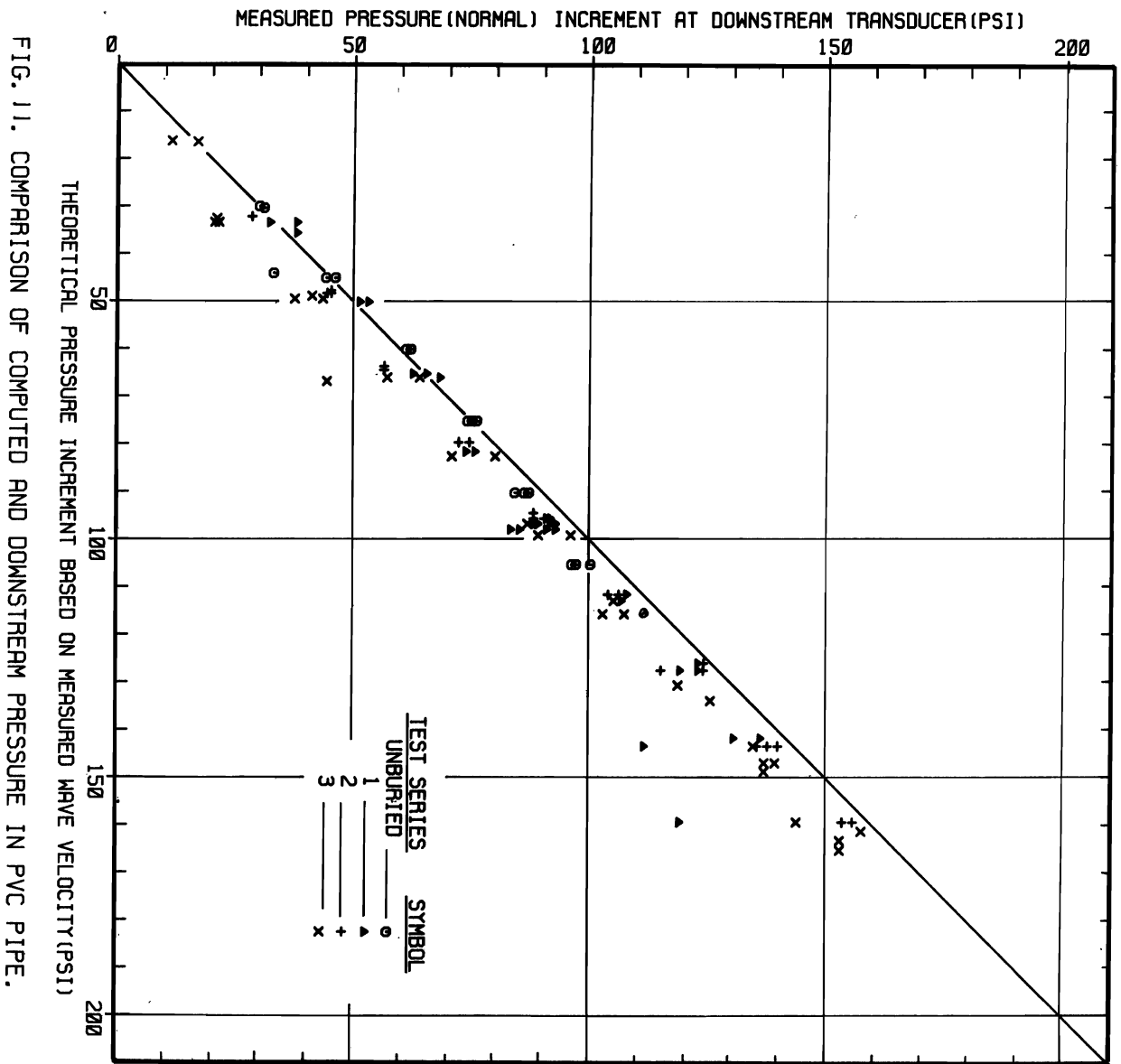


FIG. 11. COMPARISON OF COMPUTED AND DOWNSTREAM PRESSURE IN PVC PIPE.

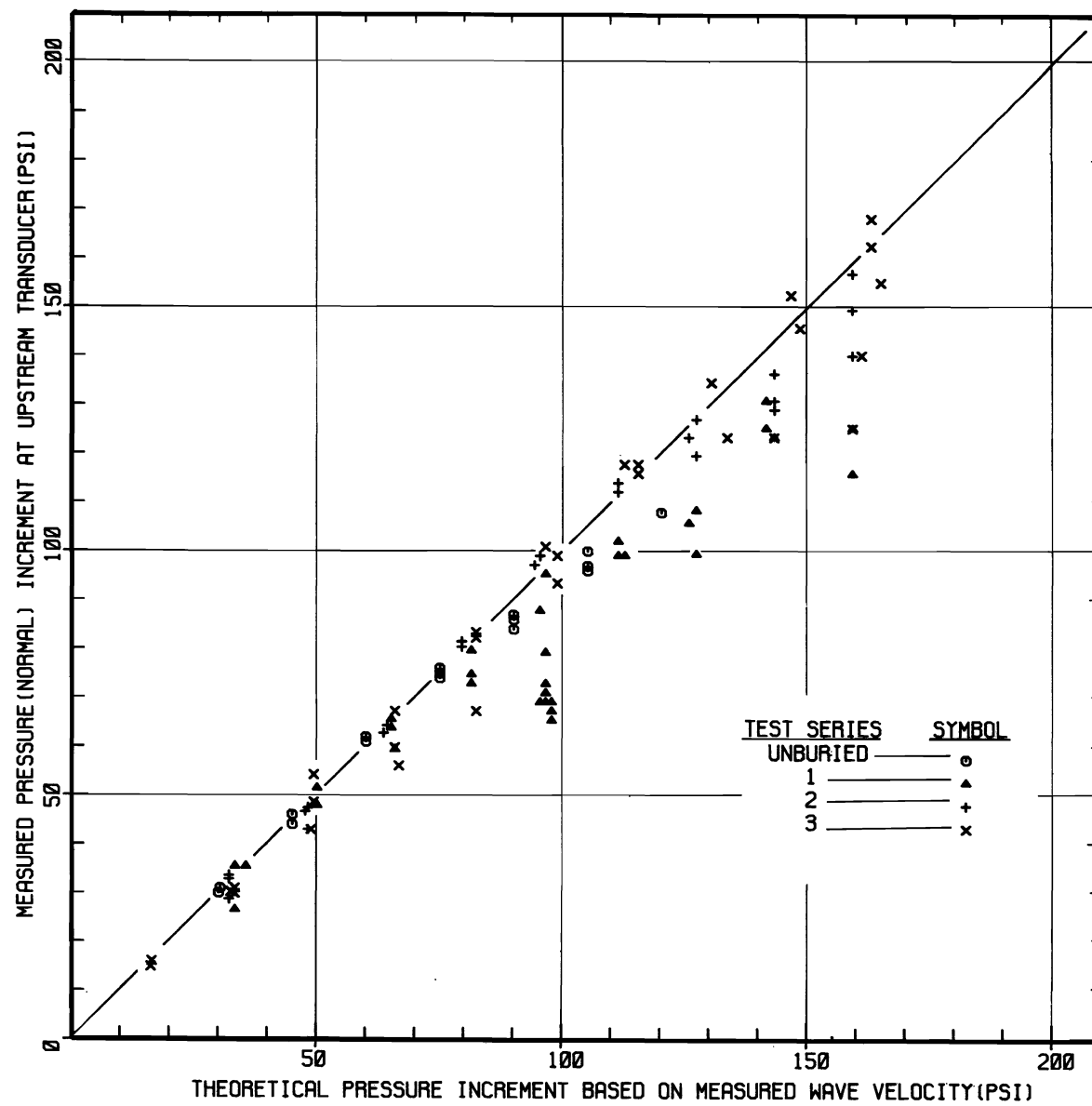


FIG. 12. COMPARISON OF COMPUTED AND UPSTREAM PRESSURE IN PVC PIPE.

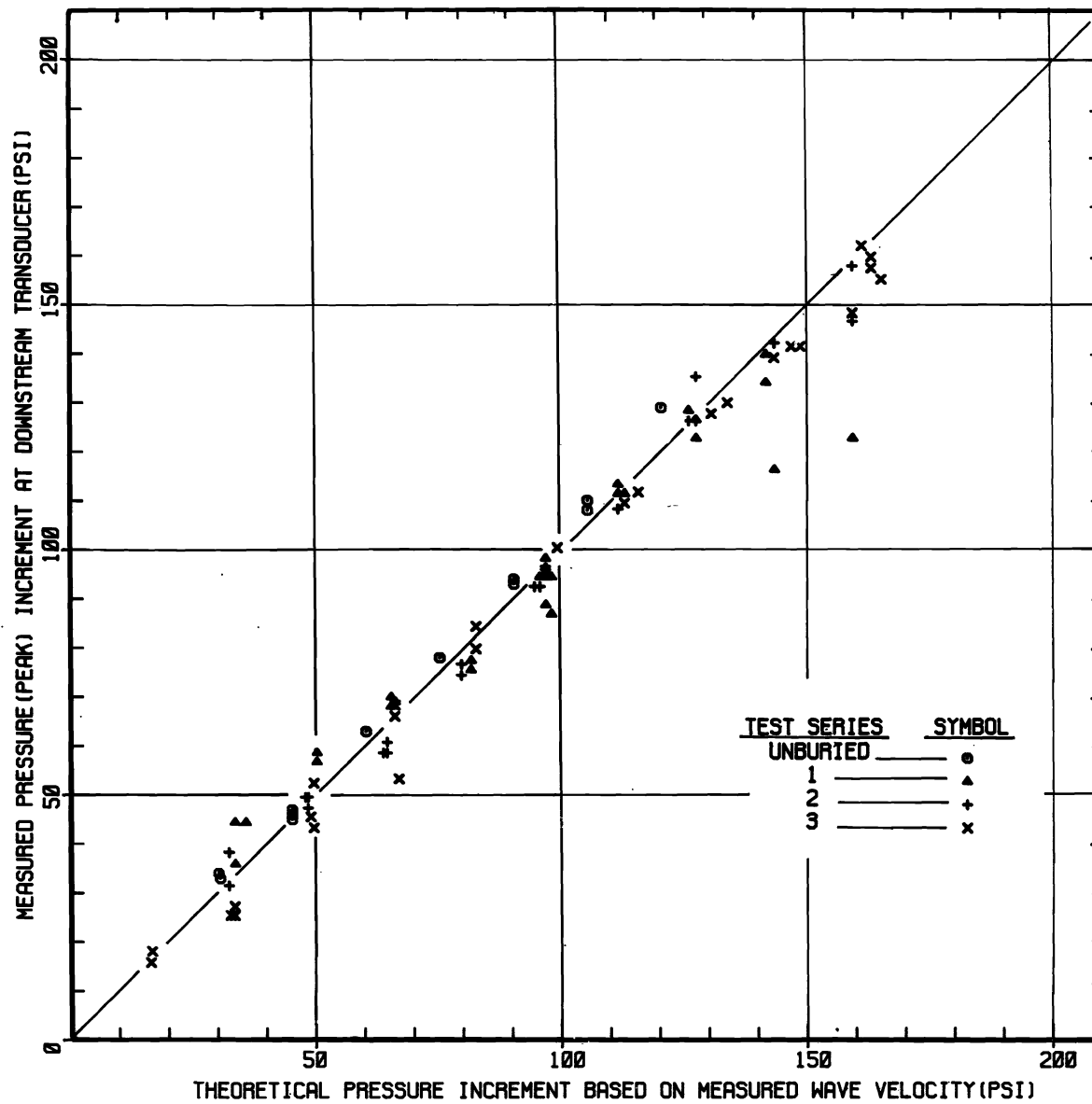


FIG. 13. COMPARISON OF COMPUTED AND PEAK DOWNSTREAM PRESSURE IN PVC PIPE.

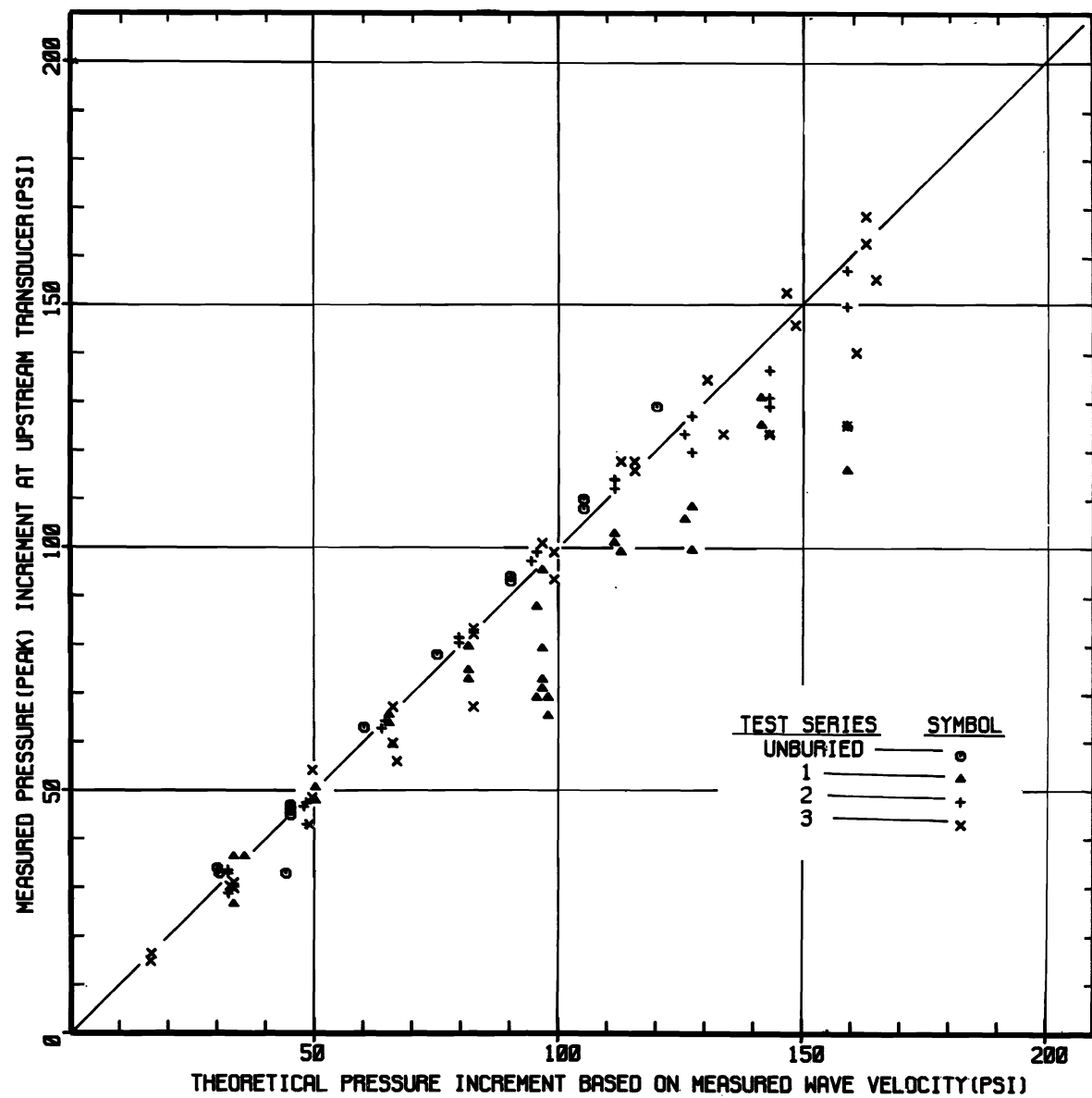


FIG. 14. COMPARISON OF COMPUTED AND PEAK UPSTREAM PRESSURE IN PVC PIPE.

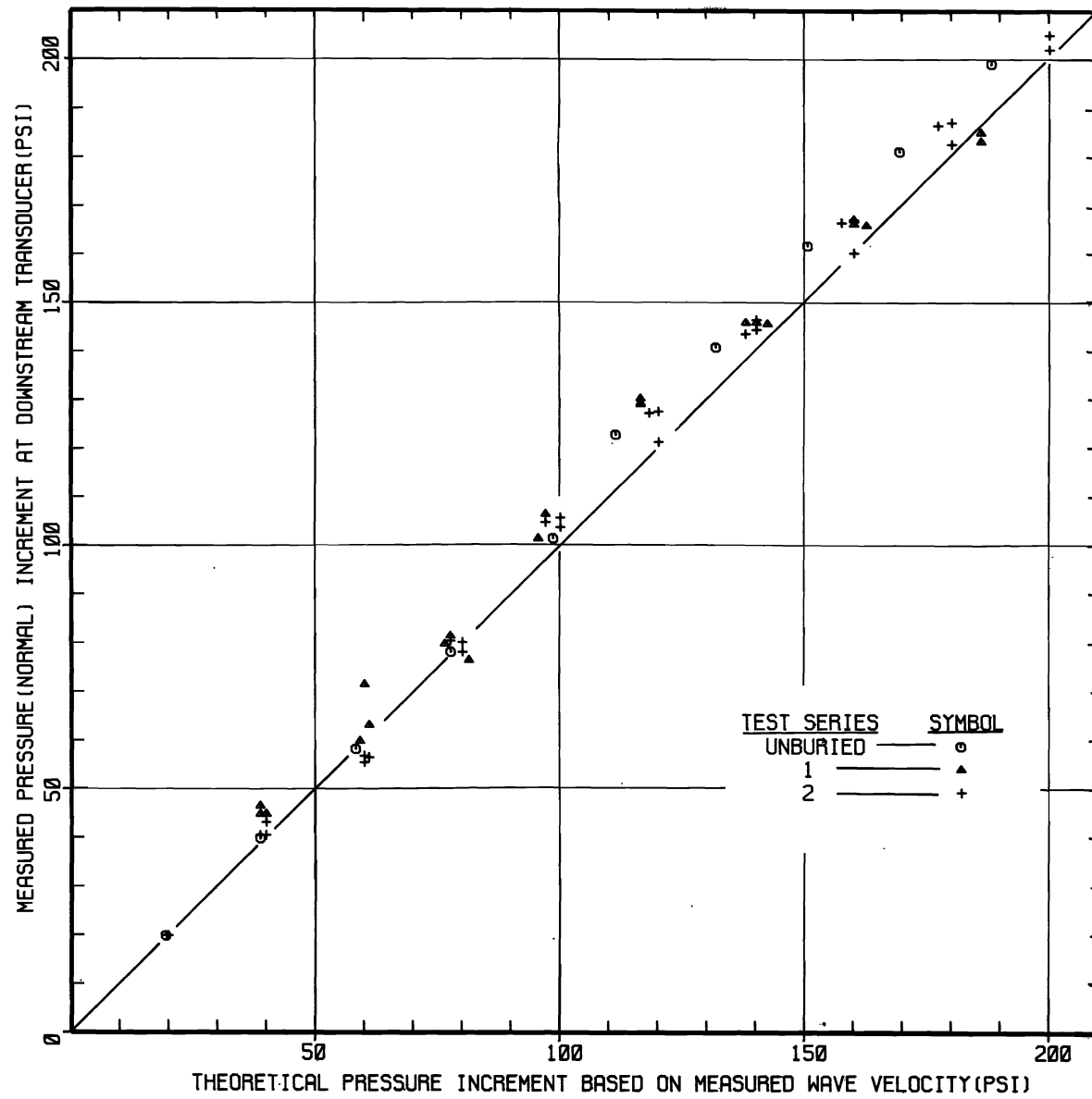


FIG. 15. COMPARISON OF COMPUTED AND DOWNSTREAM PRESSURE IN PERMASTRAN PIPE.

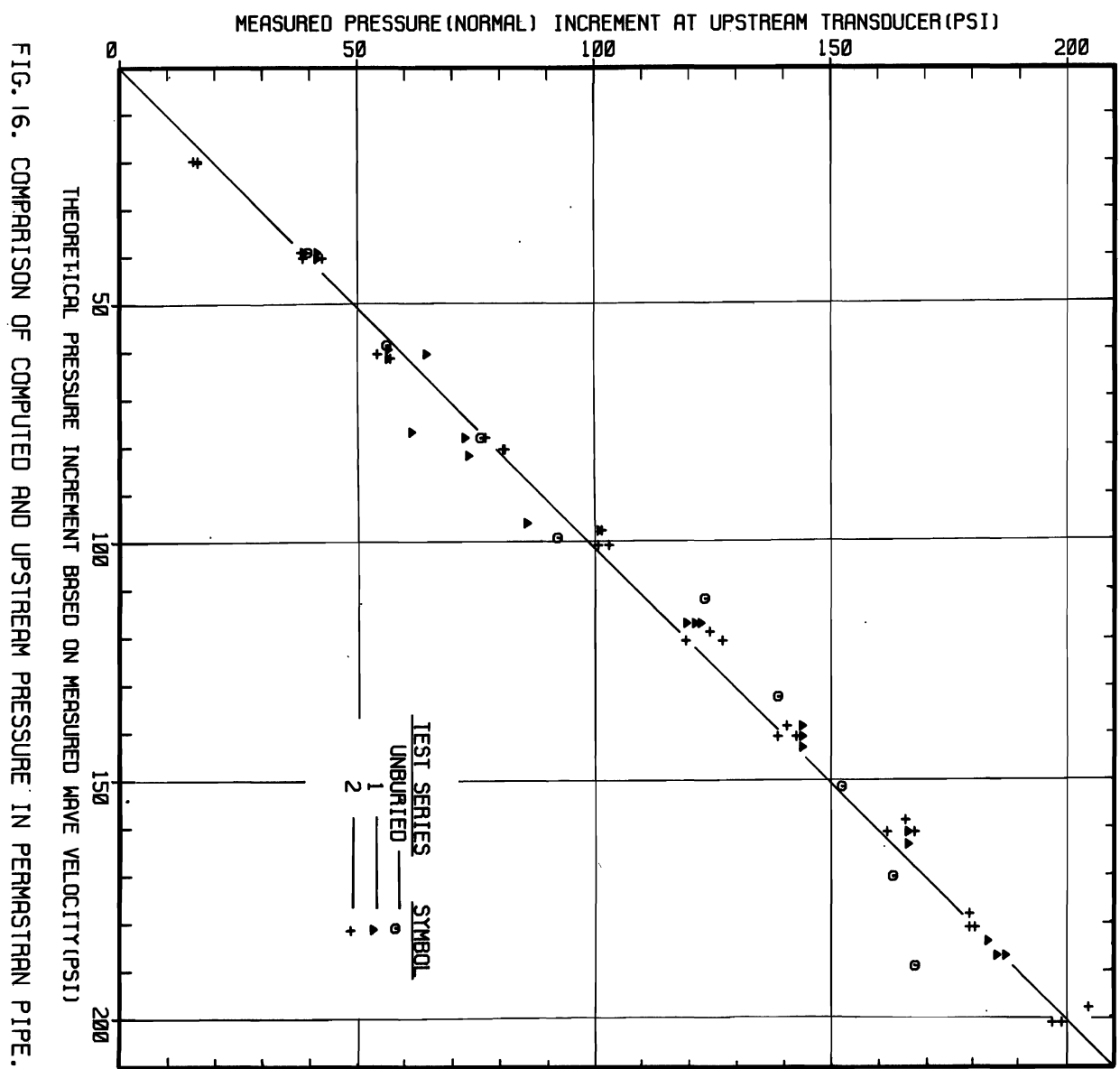


FIG. 16. COMPARISON OF COMPUTED AND UPSTREAM PRESSURE IN PERMASTRAN PIPE.

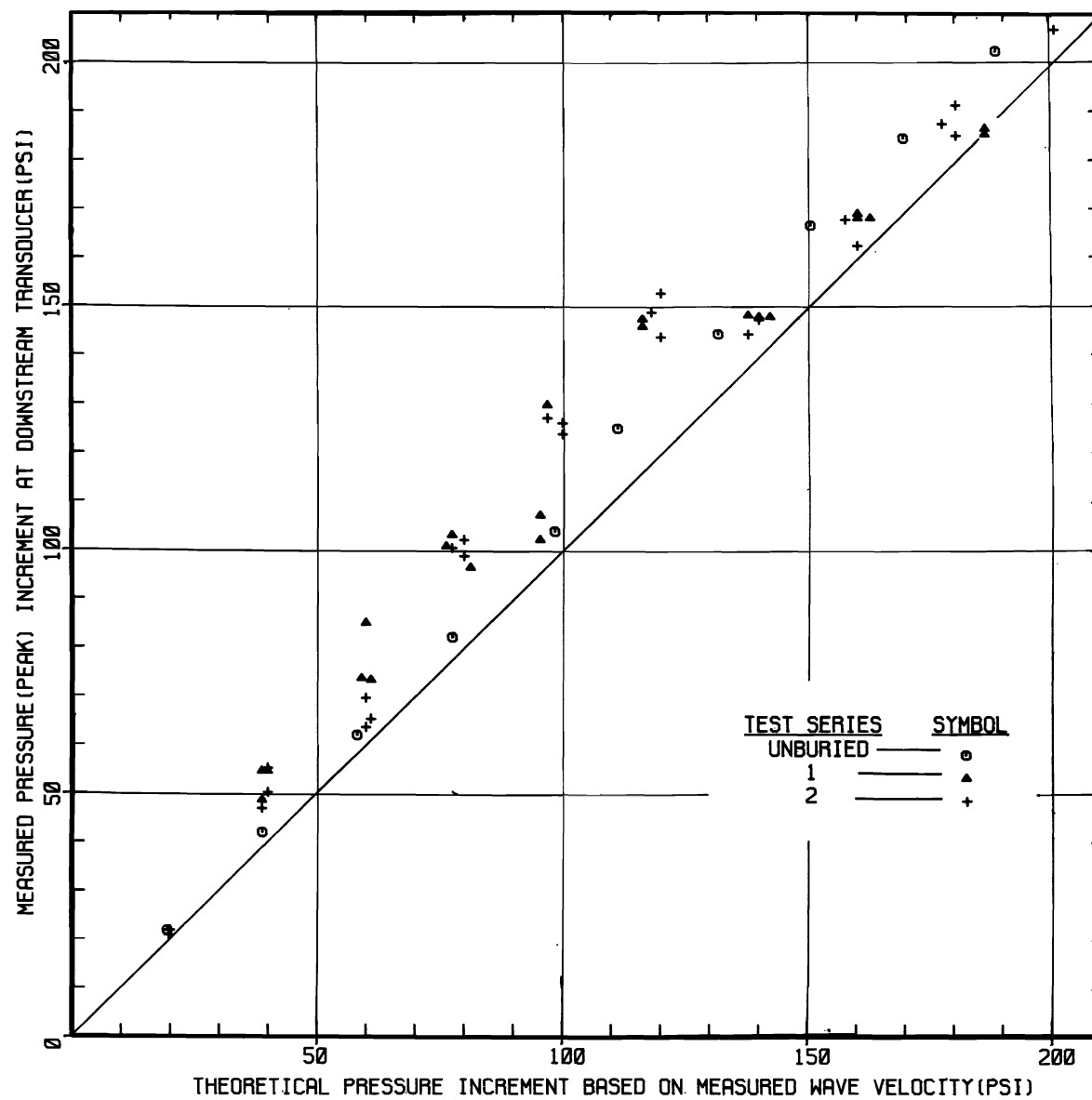


FIG. 17. COMPARISON OF COMPUTED AND PEAK DOWNSTREAM PRESSURE IN PERMASTRAN PIPE

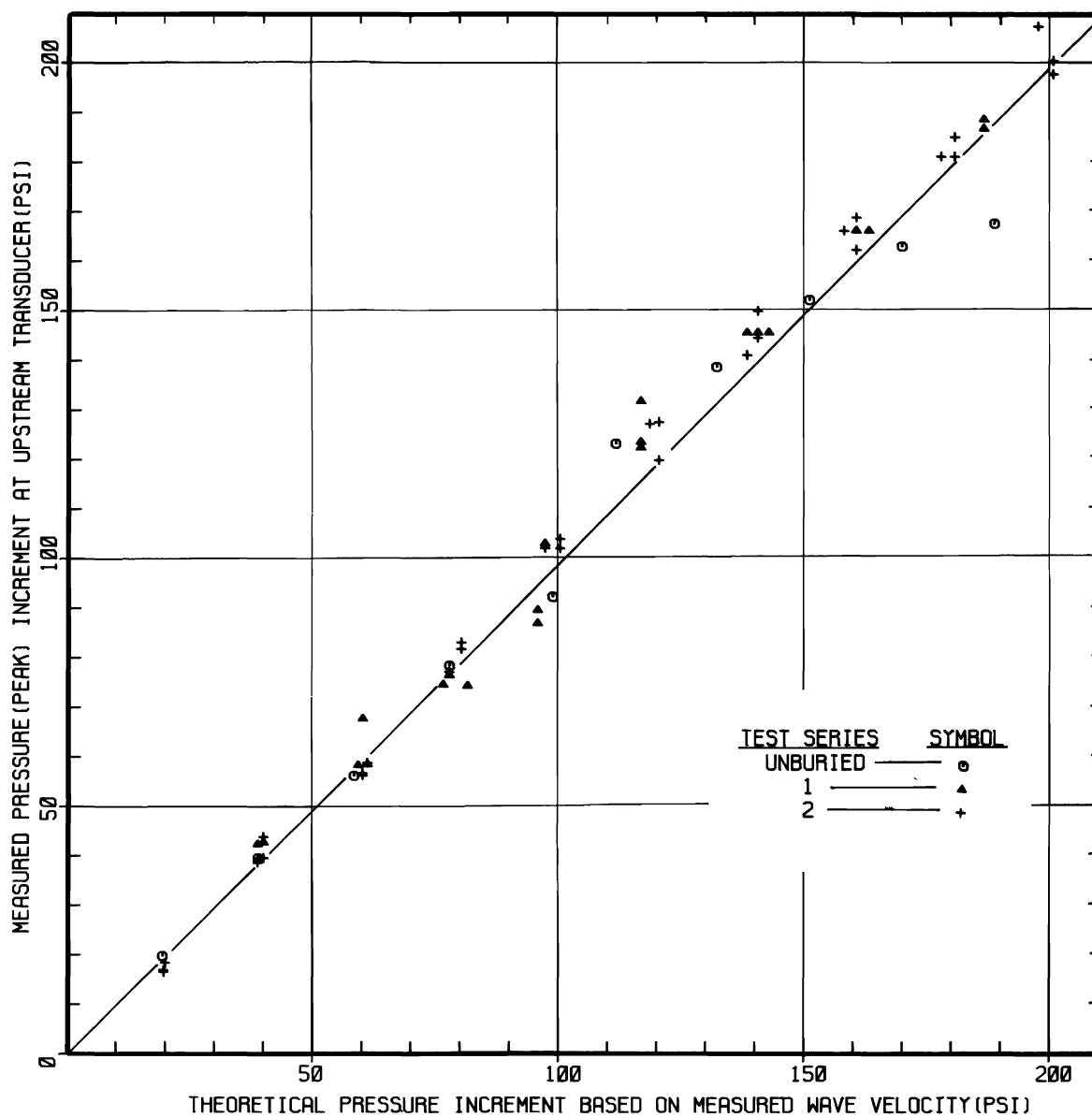


FIG.18. COMPARISON OF COMPUTED AND PEAK UPSTREAM PRESSURE IN PERMASTRAN PIPE.

to water hammer also agree reasonably well with theoretical values.

The influence of burying the PVC pipe in a well compacted fill increases the velocity approximately 5 to 10 percent. The wave velocity in buried PERMASTRAN® pipe is increased by approximately one-half this amount, or from 2 to 5 percent.

REFERENCES

- Olsen, Reuben M. (1966), Engineering Fluid Mechanics 2nd Ed., International Textbook Co., Scranton, Penn.
- Streeter, Victor L. and E. Benjamin Wylie (1967), Hydraulic Transients, McGraw-Hill Book Co., New York.
- Watters, Gary Z. (1971), "The Behavior of PVC Pipe Under the Action of Water Hammer Pressure Waves," PRWG-93, Utah Water Research Laboratory, Utah State University, Logan, Utah.

APPENDIX A

COMPUTER SOLUTION OF WATER HAMMER

The computer program used to solve the water hammer problem is based on the following assumptions:

1. The velocity of the pressure wave, resulting from transient flow conditions at a point, is a constant and known.
2. The average velocity of flow in the pipe is very small in comparison with the velocity of the pressure wave.
3. The flow is described adequately by one-dimensional equations.

Under these assumptions, the unsteady continuity and momentum equations can be combined to give the following equations which are in a form for numerical solution by the method of characteristics (see Streeter and Wylie, 1967, Chap. 3).

$$\left. \begin{aligned} \frac{g}{a} \frac{dH}{dt} + \frac{dV}{dt} + \frac{fV|V|}{2D} &= 0 \\ \frac{dx}{dt} &= a \end{aligned} \right\} \begin{array}{ll} C^+ & \text{. . . (A-1)} \\ & \text{. . . (A-2)} \end{array}$$

$$\left. \begin{aligned} -\frac{g}{a} \frac{dH}{dt} + \frac{dV}{dt} + \frac{fV|V|}{2D} &= 0 \\ \frac{dx}{dt} &= -a \end{aligned} \right\} \begin{array}{ll} C^- & \text{. . . (A-3)} \\ & \text{. . . (A-4)} \end{array}$$

in which g is the acceleration of gravity (L/t^2), a is the velocity of the pressure wave (L/t), H is the hydraulic head (L), V is the average velocity of flow in the pipe (L/t), D is the pipe diameter (L), f is the Darcy-Weisbach friction factor, t is the time, and x is the space coordinate in the direction of the pipe axis (L). Equation A-1 is valid only along the positive characteristic whose slope is defined by Eq. A-2. Likewise, Eq. A-3 is valid only along the negative characteristic defined by Eq. A-4.

Approximating the derivatives in Eqs. A-1 through A-4 by appropriate differences and combining the results lead to the following commonly used finite difference equations for advancing the flow velocity and hydraulic head, respectively, through one time increment.

$$V_i^{j+1} = 1/2 \left[V_{i-1}^j + V_{i+1}^j + \frac{g}{a} (H_{i-1}^j - H_{i+1}^j) - \frac{f \Delta t}{2D} \left(V_{i-1}^j \left| V_{i-1}^j \right| + V_{i+1}^j \left| V_{i+1}^j \right| \right) \right] = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (A-5)$$

and

$$H_i^{j+1} = 1/2 \left[H_{i-1}^j + H_{i+1}^j + \frac{a}{g} (V_{i-1}^j - V_{i+1}^j) - \frac{a}{g} \frac{f \Delta t}{2D} \left(V_{i-1}^j \left| V_{i-1}^j \right| - V_{i+1}^j \left| V_{i+1}^j \right| \right) \right] = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (A-6)$$

in which the superscript j denotes the time step, i.e., $j = t/\Delta t + 1$, and the subscript i denotes the number of the x grid, i.e., $i = x/\Delta x + 1$.

At the upstream end of the pipe ($x = 0$), the assumption has been made that a reservoir exists. This assumption does not actually represent the true conditions of the tests which consist of an 8 inch pipe preceded by a number of bends, valves and transitions which eventually lead to the pump which has different characteristic curves depending upon its rotational speed. Because of the complexity of building all of these features into the computer program, and also because the main concern of the study is measurements of the pressure wave velocities and maximum resulting pressure increases, the upstream reservoir assumption has been used in the computer solution. The reservoir condition leads to the finite difference equations:

$$H_1^{j+1} = H_1^j = H_0 \text{ (constant)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (A-7)$$

$$V_1^{j+1} = V_2^j + \frac{g}{a} (H_1^{j+1} - H_2^j) - \frac{f \Delta t}{2D} V_2^j \left| V_2^j \right| \quad . \quad . \quad . \quad (A-8)$$

At the down stream end of the pipe a programmed valve closure condition has been used in the computer solutions. The times required for the gate valve to move the first 1/2 inch the next 3 inches and finally the last 1/2 inch were recorded for each of the tests carried out on the buried PVC pipe. The rate of valve close was specified by input data giving the position of the gate of the valve and the time at which it had this position. The tests on the PERMASTRAN[®] pipe also gave data that fit well into this description. The valve was represented by a circle sliding downward within a circle of the same diameter (see Fig. A-1). With this representation, the area through which the flow can pass is given by:

$$A_v = \frac{D^2}{2} (\theta - 1/2 \sin 2\theta) - \frac{\pi D^2}{4} \quad . \quad . \quad . \quad . \quad . \quad (A-9)$$

The angle θ is related to the vertical position of the valve Y by the equation,

$$\cos \theta = \frac{Y}{D} - 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (A-10)$$

Since the same flow passes through the valve opening as through the pipe immediately upstream from the valve

$$C_d^{j+1} A_v^{j+1} \sqrt{2g H_N^{j+1}} = V_N^{j+1} \frac{\pi D^2}{4} \quad . \quad . \quad . \quad . \quad (A-11)$$

in which C_d^j is the discharge coefficient for the valve in its position at the time corresponding to j and the subscript N denotes the end of the pipe.

By dividing Eq. A-11 by itself, but with $j = 1$ leads to the following equation:

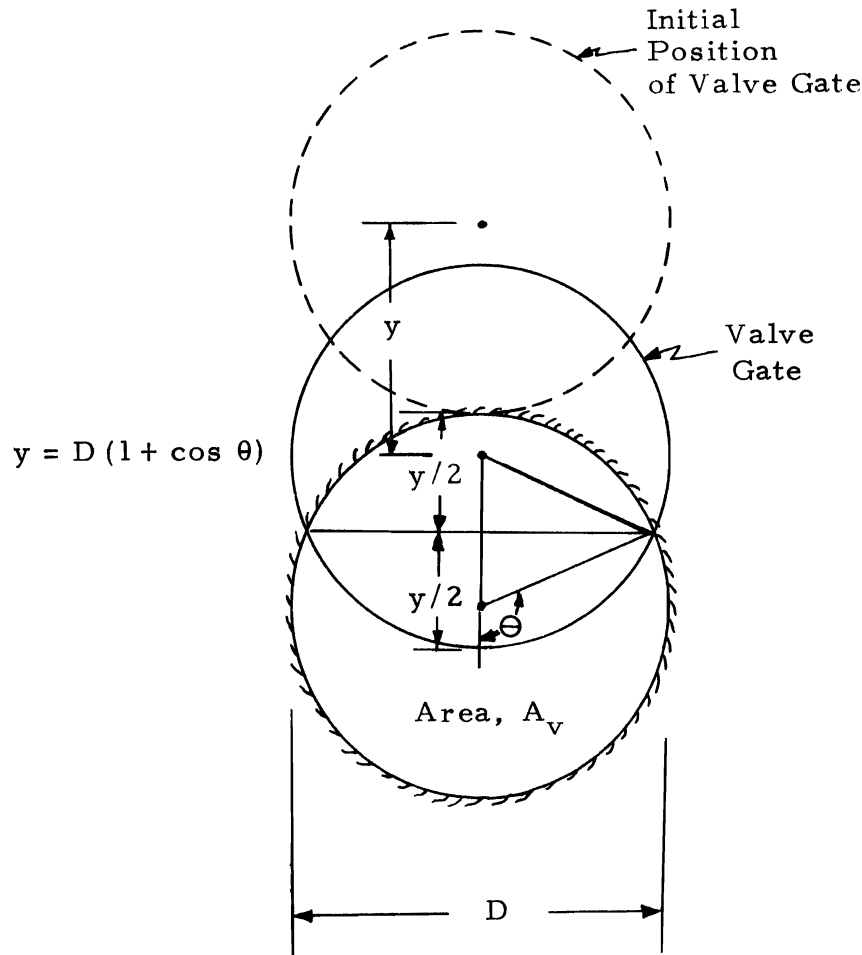


Fig. A-1. Schematic diagram of gate of valve moving down through valve opening.

$$V^{j+1} = V_o C_4^{j+1} \sqrt{\frac{H^{j+1}}{H^1}} \quad \dots \quad (A-12)$$

in which

$$C_4^{j+1} = \left(\frac{C_d^{j+1}}{C_d^1} \right) \left(\frac{A_v^{j+1}}{A_v^1} \right) \dots \quad (A-13)$$

Solving Eq. A-8 simultaneously with the finite difference equation from Eq. A-1 leads eventually to the following two equations:

$$V^{j+1} = -\frac{C_5^j}{2} + \sqrt{\left(\frac{C_5^j}{2}\right)^2 + C_3^j C_5^j} \quad . \quad . \quad . \quad . \quad (A-14)$$

and

$$H_i^{j+1} = \frac{a}{g} (C_3 - V_i^{j+1}) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (A-15)$$

in which

$$C_3^j = V_{N-1}^j + \frac{g}{a} H_{N-1}^j - \frac{f V_{N-1}^j |V_{N-1}^j|}{2D} \Delta t$$

$$C_5^j = \frac{(C_4^j)^2 (V^{(1)})^2}{H^{(1)}} - \frac{g}{a}$$

APPENDIX B
SAMPLE OF VISICORDER CHARTS

On the visicorder charts the time increment between consecutive vertical lines is 0.01 seconds. The distance between consecutive horizontal lines on the original charts equals 0.1 inch with each fifth such line heavier to make 1/2 inch intervals.

